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AMERICAN SOCIETY OF CIVIL ENGINEERS
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THE RÔLE OF THE ENGINEER IN AIR SANITATION
A SYMPOSIUM

Note.—Presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 20, 1938. Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by March 15, 1940.

A GENERAL SURVEY

By Earle B. Phelps, Esq.[1]

Synopsis

Air sanitation may be defined as the control of man's atmospheric environment, for the purpose of meeting the essential physiological requirements, and of avoiding abnormal and harmful conditions. Such a definition automatically places air sanitation within the field of Public Health Engineering, its unique feature being that the fundamental data are of the biological sciences, and its primary objective the preservation of health through environmental sanitation.

The ventilating engineer has long shown his ability and expressed his willingness to modify air to meet any specific requirements, just as the structural engineer can readily design a sewage disposal plant to perform any required physical duty. In each case, however, the requirements must be established through biological investigation, and their ultimate expression may not be in common engineering terms. Nowhere is this unique feature of Public Health Engineering better illustrated than in the field of air sanitation, in which one is forced, at the very outset of any general discussion, to depart entirely from engineering as such, and to turn one's attention to the essentials of a sanitary air environment. It is this field of inquiry—more properly termed "air hygiene"—that the writer attempts to explore herein, with no apology for its obvious inadequacy as to detail or incompleteness as to scope, and with only such occasional reference to the complementary engineering or control measures as seem to be indicated.

The Out-of-Doors Atmosphere

Man's relation to his atmospheric environment begins with the outdoor atmosphere—the much discussed weather. Despite Mark Twain, people are really doing something about the weather. By means of clothing, umbrellas, shut-in cars, and other sheltering devices they do maintain, even in their travels, a kind of personal weather-environment; and, when they confine the weather within walls, they do with it as they will. However, in a general sense, when people go outdoors they still meet "Nature in the raw." The only significant and successful modification of the open air is its pollution by smoke, dust, and gases. The extent and harmful result of such pollution has long been a matter of investigation, discussion, and attempted control. The direct health implications are many, as to possibilities, but few as to definite evidence. Occasionally, however, a successful large-scale experiment is performed.

[1] Prof., Sanitary Science, Coll. of Physicians and Surgeons, Columbia Univ., New York, N. Y.

An example of gross atmospheric pollution and of widespread damage to life is the famous "Fog Disaster" of the Meuse Valley in Belgium in 1930. Over a period of three days there were several thousand acute pulmonary attacks resulting in about 60 deaths. Cattle, birds, and even rats were killed. Various explanations, mostly designed to exonerate the industries, have been suggested. Some evidence indicates fluorin poisoning resulting from the use of fluorid in the steel and certain other industries.

Over a considerable area in Staten Island, N. Y., investigation has disclosed a sufficient concentration of sulfur dioxide in the air to cause much throat irritation and coughing, and definite damage to garden crops. The worst conditions are found during humid and foggy weather, with light winds from the direction of the industrial plants several miles away in New Jersey.

A comprehensive survey of conditions in New York City has been reported by Sol Pincus, Assoc. M. Am. Soc. C. E., and A. C. Stern.[2] Briefly, the survey shows the extent of air pollution as measured by total soot-fall, classified into soluble and insoluble matter. The former is analyzed further for sulfate, chloride ammonia, and ash, and the latter for tar, combustible matter, and ash. Control stations were established in outlying areas. The air was examined for various gases, sulfur dioxide (SO_2), ammonia (NH_3), and carbon dioxide (CO_2), and for dust and pollen.

The extensive studies of the Mellon Institute at Pittsburgh, Pa., supported by observations from numerous other places, show a marked interference with normal solar radiation, especially in the short-wave end of the visible spectrum and in the ultra-violet. Sunlight is the recognized dominant etiological factor in rickets. The frequency of occurrence and the severity of cases is maximum in the industrial cities, and minimum in the rural areas. Furthermore, normal sunshine possesses marked germicidal properties which are maximum in those rays that are most reduced by smoke. In so far as this process of natural, sunshine disinfection is of value in reducing the prevalence of infectious diseases, smokiness is harmful.

The breathing of dusty air, more or less continuously, may result in interference with normal respiratory functions, in a merely mechanical way, or by setting up specific reactions. The Pittsburgh investigations have shown a reasonably consistent correlation between smokiness and pneumonia in various parts of the city. The investigators attribute the result to mechanical interference with lymphatic drainage—whereby lungs ladened with soot are poorly equipped to oppose infection—rather than to interference with the normal germicidal properties of sunlight.

Finally, an increasing knowledge of hay fever and other allergic manifestations have led to a considerable study of pollens in the air and to a definite program of eradication. An excellent example of this is the work done by the Department of Health of New York City. Regular air examinations were made at many stations over the city areas, and the pollens were counted and identified as to botanical genus. These data were used to direct the work

[2] "A Study of Air Pollution in New York City," by Sol Pincus and A. C. Stern, *American Journal of Public Health*, 27, April, 1937, p. 321.

against ragweed, which for many years has been a routine activity in this department, as in many other health departments.

It must be freely admitted, however, that the economic argument against air pollution thus far has been more powerful than the hygienic argument in stimulating the clean-air movement. Here the issue is clear-cut. Abatement in many cases results in ultimate economy to the offending power plant, as well as material advantage to the households, shopkeepers, and average citizens.

The Indoor Air

Within doors, air sanitation is much more clearly defined. It is convenient to separate industrial conditions from those due merely to human occupancy. For the most part, the latter are uniform and predictable. They form the basis of the modern arts of heating, ventilation, and air conditioning. Industrial conditions cover the widest possible range, in both the quantitative and the qualitative sense.

Any effective control of the indoor atmospheric environment must first satisfy the requirements of human physiology. It has long been known, for example (and may be recorded as the first physiological principle), that the harmful effects of so-called "vitiated" air are due to its physical rather than to its chemical properties. Confined air first fails to function as a suitable environment by its reduced capacity to remove the waste heat from the human body. Much later, if at all, it fails in its capacity to support respiration.

From an engineering viewpoint, the problem of the removal of excess heat from a given body is an exceedingly simple one; to the physiologist, dealing with the human body, it is most complex. After years of investigation, during which many "blind alleys" have been explored and some most promising paths have had to be retraced, the situation at which physiologists have now arrived is essentially as follows:

Heat leaves the body by various avenues—by conduction, convection, and evaporation of moisture, and, from the skin, by radiation. Thermal equilibrium, in a physiological sense, is much more than a mere numerical balance between heat production and heat removal. It is a most delicate balance among the various avenues of heat loss, intimately and automatically connected with each other and with the thermal gradients of the body—the whole being responsible for that sense of well-being known as comfort, as well as for certain reactions and conditions definitely associated with resistance to infection.

Prof. C. E. A. Winslow and his colleagues at Yale University, New Haven, Conn., have stressed in their work[3] the idea of partitional calorimetry, the calorimetry of the body being divided into its various avenues of heat loss and gain. One of the "blind alleys" of the past, exploration of which has impeded advance, has been the concept of equivalent conditions. Under this concept, an excessive rate of heat loss by one channel (evaporation, for example) can be balanced exactly by decreasing the losses in other channels. Out of this concept have come such terms as "effective temperature" in which the total of all cool-

[3] "A New Method of Partitional Calorimetry," by C. E. A. Winslow, L. P. Herrington, and A. P. Gagge, *American Journal of Physiology*, 116, August, 1936, p. 641. See also series of papers in this Journal and in the *American Journal of Hygiene*, Vol. 26, 1937.

ing effects is supposedly summed up in one item. Unfortunately for any such simplification, merely physical equivalence is not physiological equivalence. Comfort and health demand a balance among the various manners of heat loss in addition to an over-all balance between production and total loss. As the writer has stated this proposition in another place,[4] "An arctic explorer, in his furs, and a swimmer in the surf of Atlantic City may both be disposing of the same number of calories per hour, but one would hardly class the two environments as equivalent."

Just what does constitute the most favorable indoor atmosphere as regards its purely physical nature? Temperature, humidity, and movement are recognized as among the primary variable properties associated with thermal equilibrium in the partitional sense. To these one must add an important feature not at all related to the physical properties of the air itself—namely, the temperature of the enclosing walls. L. B. Aldrich has shown[5] by direct measurement that radiation to surrounding walls, under normal indoor conditions, may account for nearly one half of the body's heat loss. Obviously, various adjustments of these four environmental conditions will result in corresponding adjustment of the internal conditions and a readjustment of the manner of heat loss. It is this finer adjustment rather than an over-all balance that forms the basis of present-day research and that promises to clarify the long studied, but elusive, question stated at the beginning of this paragraph.

The problem has been approached largely from the empirical side. The rather crude but suggestive data of the older physiologists have but slowly filtered into the practice of ventilation. Old errors have better withstood the process of infiltration than has the truth; hence the carbon-dioxide basis of ventilation standards, scarcely discarded to this day. In the "cut-and-try" method of advance, the criterion of "comfort" has been generally used. In some few instances, notably in recent studies of air conditioning, the more fundamental criteria of incidence of respiratory disease, working efficiency, absenteeism, labor turnover, etc., have been reported. The distinction is important. Comfort is largely a matter of adaptation. Undoubtedly, residents in the United States are adapted to an unhygienic indoor temperature, as foreign visitors never fail to point out. On the other hand, the comfort test has indicated certain unknown factors not correlated with the physical conditions ordinarily measured. On the whole the empirical approach has definitely advanced the art.

The experimental approach has been less aggressive, more spasmodic, but of definite value. It has at least exposed certain of the fetishes—the carbon-dioxide and 30-cu-ft-per-min fallacies, for example—and like any good scientific work has corrected some of its own earlier errors, such as the relation of humidity to cooling during the winter heating seasons. It has drawn attention to an important principle—that the external conditions must be adapted to the internal. Different air conditions are necessary to meet the variations of age,

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[4] "Environment in Relation to Public Health," by Earle B. Phelps, *The DeLamar Lectures*, 1926–27, p. 192, Baltimore, 1927.

[5] "A Study of Body Radiation," by L. B. Aldrich, Smithsonian Miscellaneous Collections, 81, No. 6, 1928.

and of occupational activity. Finally, those working on the experimental approach are undertaking a detailed study of the physiological reactions involved in body cooling, under all combinations of the environmental variables, and are doing this for the first time as a contribution to air hygiene research rather than to general physiology.

Air Bacteriology

An entirely distinct phase of air sanitation—that of air-borne infection—has been brought to the attention of students, with new emphasis, by the work of W. F. Wells [6] who has developed an improved method for bacteriological air examination, and has investigated the bacterial content of air in many places. He revives, with new evidence, the discarded views of earlier days and suggests that air bacteriology must begin, as did water bacteriology, with quantitative and qualitative examinations, and the search for significant correlations with total numbers of bacteria or with specific indicators of harmful contamination.

Summarizing their work W. F. Wells and M. W. Wells[7] show that mouth spray, in part, settles to the ground as droplets; but that, in larger part, it evaporates, leaving nuclei with their contained bacteria capable of floating in the air like tobacco smoke. Various organisms, characteristics of the human respiratory tract (including certain pathogenes and viruses), are capable of surviving, when thus suspended in air, for periods of from an hour to at least two days. Organisms sprayed into the air of the basement of a three-story building were later recovered throughout the corridors of the building. Influenza virus sprayed into an experimental chamber remained suspended and viable for at least an hour. The number of alpha streptococci found in the air of various places corresponds to the density of occupation. The possibilities of aerial infection over a considerable area, as between hospital wards, and over a length of time, is thus manifest. Sterilization of air by ultra-violet light is shown to be effective, and its feasibility upon a working scale has been demonstrated.

Leon Buchbinder, M. Soloway, and M. Solotorowsky[8] have reported the results of examinations of more than 7,000 samples of air from schools, subways, theaters, etc., in New York, N. Y. They find a close correlation between the numbers of alpha hemolytic streptococci and conditions of occupancy and of ventilation. The numbers were higher in schools than in the more congested but better ventilated subway cars; they were higher in unventilated, than in well ventilated, theaters; and they were least in the open parks.

Sanitarians are beginning, therefore, to accumulate data for a bacterial measure of air pollution, and of ventilation and air cleansing efficiency. Although it is too soon to attempt any generalization it may be said fairly that airs may now be compared as to their relative contamination by organisms of the human respiratory tract, and that engineers are now in a position to discuss methods of ventilation and of air cleansing from a bacteriological viewpoint, and

[6] "Apparatus for the Study of the Bacterial Behavior of Air," by W. F. Wells, *American Journal of Public Health*, 23, January, 1934, p. 58.

[7] "Air-Borne Infection," by W. F. Wells and M. W. Wells, *Journal*, Am. Medical Assoc., 107, November 12, November 28, 1936, pp. 1698, 1805.

[8] "Alpha-Hemolytic Streptococci of Air," by Leon Buchbinder, M. Soloway, and M. Solotorowsky, *American Journal of Public Health*, 28, January, 1938, p. 61.

thus to supplement the physiological findings as to optimum conditions for thermal equilibrium, with bacteriological findings as to permissible limits of contamination.

Industrial Conditions

The manifold problems of air sanitation in industry are so diverse as to constitute a distinct field of study which one cannot hope to explore in a brief survey such as this. Generally, they are of the same nature as the problems of normal air conditioning but upon an exaggerated scale, dealing with extremes of heat, humidity, cold, dustiness, and chemical pollution. Many of these conditions are inherent and unavoidable, as in a steel mill or a refrigerating plant. Others, such as the hot, humid air of deep mines, may be remedied only at great cost. The airs of working places, polluted with dusts, fumes, vapors, and gases, are more readily controlled, and a brief glance at some of the types of problem involved may be of interest.

Passing over these industrial conditions in which dust is objectionable primarily because of its concentration and mechanical effects upon the respiratory tract, two typical dusts or fumes of specific injurious nature may be noted. In mining, quarrying, and rock excavation, the breathing of the dust which results from drilling quartz, or certain silicate rock, leads in time to a pathological condition of the lung tissue, known as silicosis. Pulmonary tuberculosis later develops in many of these cases. Adequate ventilation, wet-drilling methods, or protective masks are used to offset this danger.

Finely divided metallic oxides, zinc, lead, etc., arise from the hot or molten metals in many metallurgic processes. Breathed in sufficient concentration they give rise to a condition known as metallic fever. In addition, a specific toxic effect of the metal itself may also be experienced, as in lead poisoning.

Toxic vapors and gases arising from innumerable chemical operations each presents a separate problem, including a physiological study of toxicity and permissible limits in the air. The former extensive use of benzol, for example, in spray painting is typical. Rather widespread prevalence of benzol poisoning resulted before the seriousness of the situation was realized and other solvents substituted. As an indication of the extent of this field, the toxicology and pharmacologic reactions of 32 different volatile substances in the single chemical group of halogenated hydrocarbons used in industry have been studied. Ventilation based upon good engineering practice is the general preventive measure. Before the problem is ready for the ventilation engineer, however, two sets of data are required. Specific gravity, temperature, and other physical data, together with a knowledge of the industrial process itself, determine the best type of ventilating procedure and the proper location of hoods, duct inlets, etc. In addition, ventilation practice must be based upon adequate physiologic knowledge of human tolerance and permissible concentrations.

Vehicular tunnels, such as the Holland Tube between New York and New Jersey, present a unique situation, approximating industrial conditions. In this case a study was made of human tolerance to carbon monoxide, upon a concentration, exposure-hour basis, and of the average rate of production of the

gas by various types of motor vehicles. These determined the necessary rate of air supply and exhaust. All this, be it noted, preceded the study of the actual ventilation problem itself. The carbon monoxide contamination of the general atmosphere by motor vehicles, and especially of the air within heavy buses by their own exhaust gases, was studied by Messrs. Pincus and Stern,[2] and has received some study elsewhere.

From the foregoing brief survey it is evident that a field of knowledge and of administrative control is outlined herein which is much broader than that customarily included within engineering. It will readily be discerned, however, that this is not a novel situation. The design and operation of a water, or a sewage, treatment plant or of a milk handling and pasteurizing plant, the control of mosquitoes, and many similar projects engage engineering ability and utilize engineering knowledge to accomplish certain purposes that are expressed only in terms of biology and chemistry. It is believed that the public health engineer must be prepared, with the necessary collaboration from the associated sciences, to assume ultimate responsibility and control in all such cases. The purpose of this paper is to indicate the scope and basis of air sanitation—that is, the control of the atmospheric environment—regarded as an engineering project. The field is already occupied in several of its parts by various engineering specialists. A second purpose of this paper is to indicate the essential unity of this field, the interdependence of its parts, and its ultimate dependence as a whole upon the biological sciences.

Summary

Air sanitation, the control of man's atmospheric environment, is a field in which the public health engineer is called upon for guidance and in which his responsibilities are becoming increasingly evident with the growth of knowledge of the relation between that environment and human health and comfort. The successful solution of its manifold problems demands a somewhat intimate knowledge of those relations, which in turn are based upon the biological sciences, physiology, bacteriology, bio-physics, and bio-chemistry.

Pollution of the general atmosphere results from the discharge of smoke, gases, and fumes, and from industrial or household stacks. It reduces or eliminates the useful ultra-violet radiation, as well as the actual sunshine; it interferes with respiration; it leads to annoyance, minor ills, acute sickness, and even death; and it causes damage to crops, to structures, and to goods.

The problems of heating, ventilation, and air conditioning deal with the atmosphere indoors, subject, in the simplest case, merely to the effects of human occupancy. The determination of what constitutes the ideal indoor air for health and comfort is still undergoing study, but with a more concerted and direct attack by physiologists and with promise of a satisfactory outcome. The reactions of a human subject are not the mere physical reactions of an inanimate body, and the finer mechanism by which man maintains his thermal equilibrium is highly complex and interrelated. No single formula can express these relations and no gross integration of external conditions can meet them.

In modern days, the older theories of air-borne infection have been entirely rejected or held unlikely of practical usefulness. This has been due to inherent

weaknesses of the theories themselves (most of which antedate modern bacteriology), and to the identification of many other avenues of infection, such as food, drink, insects, and direct contact. Newer studies in air bacteriology have served to show the total or partial inadequacy of these avenues (in the case of the respiratory infections in particular) and the need to reexamine available data on air-borne infection. These studies are also providing a basis for the practical evaluation of methods of ventilation and air cleansing.

Industrial hygiene deals largely with the healthfulness of environmental air conditions in industrial plants. The field is a broad one covering every phase of ordinary heating, ventilating, and air conditioning, but in an exaggerated degree, and in addition many new problems of specific nature. The ill effects of the inhalation of quantities of any inert dust, the specific effects of certain dusts such as silica or hot metal fumes, the toxic effects of many volatile chemical substances such as benzol, and the ill effects of extremes of heat, cold and moisture, and of sudden changes to and from any of these, are some of the important problems involved.

In his study of methods of air sanitation, the engineer must know the possibilities of good engineering practice and combine with this knowledge a knowledge of the physical, chemical, bacteriologic, physiologic, and toxicologic properties of the special types of impurities with which he has to deal. This constitutes indeed a much broadened definition of engineering and will doubtless require, as in the case of water supplies and food sanitation, cooperative effort among specialists; but, it is to the public health engineer that the future must look for guidance and responsible control.

TYPICAL PROBLEM IN INDUSTRIAL SANITATION

By J. J. Bloomfield,[9] Esq.

Synopsis

Industrial hygiene has been defined as the science of the preservation of the health of the industrial worker. Although it has been known from ancient times that ill health and premature death were often associated with the nature of man's livelihood, it is only in recent years that studies have clearly indicated that the health of workers engaged in industry may be affected by the materials and processes used. For these reasons one of the important phases of industrial hygiene is the study of the relationship between the working environment and its effect on health and methods of control. This phase of industrial hygiene may aptly be termed industrial sanitation, and is a function coming within the province of the engineer.

It is the purpose of this paper to discuss the rôle of the engineer in industrial sanitation studies, and to indicate the trends in this field of public health.

Classification of Health Hazards

Before discussing the various problems confronting the engineer in industrial sanitation, it may be well to define the present conception of industrial health hazards. There are several classifications of health hazards in industry, and the following is only one version. Industrial health hazards may be divided into three types: Chemical, biological, and physical. Under chemical hazards is the important group of poisons, of which there are a great number and variety. L. I. Dublin and R. J. Vane [10] list about 94 groups of industrial poisons in the United States, associated with approximately 2,500 occupational exposures. Health hazards such as poisons may again be classified according to their physiological and pathological effects on the human system. Biological health hazards include the infections, such as anthrax, tuberculosis, typhoid fever, respiratory, and venereal diseases. The physical health hazards are extremely important in industry; and this classification embraces those dusts which may cause fibrosis of the lungs, accidents caused by machinery and other environmental conditions, excessive humidity, heat and cold, and abnormal atmospheric pressure.

From the foregoing suggested classification of industrial health hazards, it should be evident that the practice of industrial hygiene is primarily within the sphere of two types of workers, the physician and the engineer. It is the

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[9] Sanitary Engr., U. S. Public Health Service, Washington, D. C.

[10] "Occupation Hazards and Diagnostic Signs," by L. I. Dublin and R. J. Vane, *Bulletin No. 582*, Bureau of Labor Statistics.

physician's task to recognize the existence of those diseases associated with the working environment, while the engineer's function is to study the working conditions that may be detrimental to health, and, by precise, quantitative measurements, to determine the extent of the hazard. Once the nature and degree of the hazard have been demonstrated, it is also the engineer's problem to evolve methods for controlling or minimizing the hazardous condition, and finally to study the effectiveness of these measures.

Engineering Aspects of Industrial Sanitation

The magnitude of the problem confronting the engineer in industrial sanitation is as large as it is varied. Industrial sanitation surveys and the various engineering methods followed in controlling industrial health hazards have been discussed fully in previous publications.[11, 12] In this paper the rôle of the engineer in industrial hygiene will be demonstrated from the viewpoint of a mass attack on one particular industry. It is felt that the methodology described for a study of the hazards in this industry may be applied with slight modifications to engineering studies of other industrial problems.

In brief, engineering studies of the working environment involve the determination of the occupational exposure to such air-borne materials as dusts, vapors, fumes, mists and gases, and to substances which may cause corrosive burns or dermatoses. Industrial sanitation also includes studies of the physiology and mechanics of ventilation, illumination, exposure to extreme conditions of temperature, humidity, noise, studies of fatigue, general plant sanitation—in fact, all phases of the environment which may have some bearing on health. In addition, it is evident that in order to have a broad perspective of the entire problem, it is essential that the engineer be at least familiar with the toxicology of many of the materials used in industry and have some knowledge of industrial legislation and labor economics in general.

In the past, those responsible for industrial hygiene administration have been primarily concentrating their efforts on individual plant or workroom studies. It is evident that such procedure, when practiced in some industrial centers or states, would not result in a rapid material improvement in the environmental conditions for a large portion of the working population. Students of the problem of industrial sanitation now realize that more rapid strides may be made by a study of representative plants of an industry, the results of which may then be applied to the entire industry. A description of one of the studies conducted by the U. S. Public Health Service should serve to demonstrate some of the activities involved in industrial sanitation. The example that follows concerns primarily the application of engineering studies to the hatters' fur cutting industry and exemplifies the approach to the problem of industrial sanitation.

Methods and Instruments Used in the Study

At the time of this study thirty-six plants in the United States were engaged in the preparation of hatters' fur, employing approximately 2,000 persons.

[11] "Preliminary Surveys of the Industrial Environment," by J. J. Bloomfield, *Public Health Reports*, Vol. 48, No. 44, November 3, 1933.

[12] "Engineering Control of Occupational Diseases," by J. J. Bloomfield, *Public Health Reports*, Vol. 51, No. 21, May 22, 1936 (Reprint 1749).

Since it was not feasible to examine every worker in the industry, it was necessary to make a selection of plants. This was accomplished by a preliminary survey, which consisted of collecting such data as industrial welfare facilities afforded the workers, the general sanitation of the plant, and details concerning the processes and occupations involved in the preparation of hatters' fur. Certain other information was obtained, such as facilities for conducting physical examinations, and cooperation of employers and employees in the conduct of the study, all of which influenced the selection of the plants. In this manner it was possible to choose five plants proportionately representative of modern and old practice, as well as those which had, on visual inspection, good, fair, or poor working conditions. In addition, this preliminary survey served to give a fairly good "picture" of the entire industry.

Further preparatory steps prior to actual quantitation of the working environment consisted of making sanitary and occupational surveys of the various workrooms in the five plants. Such preliminary surveys serve as a guide for the more detailed studies which follow. The sanitary survey, for example, lists the various facilities afforded the employee in the working environment, whereas the occupational survey permits one to determine the activities involved and the particular hazards associated with each occupation. Survey forms similar to those described elsewhere[11] served as a basis for recording the essential data on the sanitary and occupational phases of this study.

The preliminary survey showed that the major occupational hazard in need of detailed investigation was the exposure to mercury vapor and dust. Certain occupations involving an exposure to fur dust made it advisable to obtain a particle count of this exposure also. For the determination of mercury vapor, use was made of a special selenium sulfide detecter[13] because of its portability, ruggedness, simplicity of operation, and the rapidity of obtaining immediate results. In the present study, mercury dust was excluded from the detecter by placing a single-thickness "paper thimble" at the point where air enters the instrument. The dust-free air then passed into the device, was preheated to a temperature of 70° C, and then allowed to come in contact with a sensitized paper. The method is based on the reaction between active selenium sulfide coated on the paper, and the vapor. Whenever mercury vapor comes in contact with the paper, it is blackened, the degree of blackness being an indication of time of exposure, concentration of mercury, and other factors that can be controlled definitely. The quantity of mercury vapor in the air can be determined by comparing the sample obtained with standards which accompany the instrument.

In order to determine the quantity of mercury dust in the air, samples were obtained with the impinger apparatus.[14] Since it was found in preliminary observations that the mercury coated fur was of a greasy texture and hence difficult to wet, it was necessary to use a 25% alcohol and water mixture as a collecting medium. Samples thus obtained were shipped to the central laboratory at Washington, D.C., where they were analyzed.[15]

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[13] "Selenium Sulfide—A New Detector for Mercury Vapor," by B. W. Nordlander, *Industrial and Engineering Chemistry*, Vol. 19, No. 4, April, 1927.

[14] "Determination and Control of Industrial Dust," by J. J. Bloomfield and J. M. DallaValle, *Public Health Bulletin No. 217*.

[15] "The Determination of Mercury in Carroted Fur," by F. H. Goldman, *Public Health Reports*, February 19, 1937 (Reprint 1804).

A Konimeter was used for the determination of the quantity of dust to which the workers were exposed.[16] In the control studies which followed, certain ventilation readings were made on the exhaust systems and these were obtained with the pitot tube.

Preparation of Hatters' Fur

The processes in the manufacture of hatters' fur may be considered roughly in three parts: (1) The preparation of the pelt; (2) the carroting and drying of the fur; and (3) the cutting or shearing of the hair from the hide. These constitute the basic steps in all hatters' fur establishments. Such differences as may be found among various plants pertain chiefly to the degree to which some departments have been mechanized.

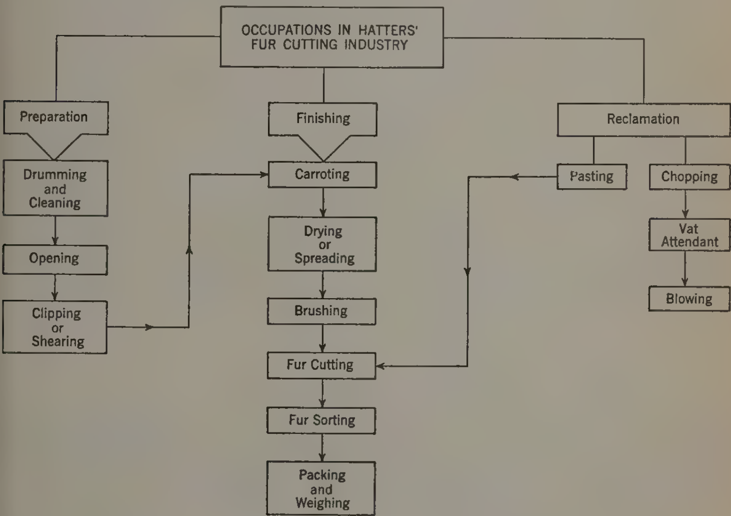

Fig. 1

Fig. 1 is a flow sheet depicting the major operations involved. Briefly, the preparation of hatters' fur consists of softening and cleaning rabbit pelts by rotating them in a drum for about one hour with a mixture of wet sawdust and sand. This softens the brittle pelt and cleans the fur. The next step is to remove the sawdust and sand from the pelts by rotating them in a wire-mesh drum. The skins are then delivered to the openers, who mount the reversed skin on an upright wooden fork and cut off the head, tail, and legs, and slit the hide. Next the long hairs are removed by a revolving cutter, designated as clipping or shearing.

[16] Final Report of the Miners' Phthisis Prevention Comm., Union of South Africa, March 10, 1919.

Up to this point the workers have been handling raw (untreated) skins. The processes which follow are associated with the use of mercury in one form or another.

In the next operation, known as carroting, a mixture of mercury nitrate and nitric acid is applied to the tips of the fur with either a hand or revolving brush. The skins are then dried either at room temperature or slightly below 140° F in mechanical driers (this is known as white carroting); or they are dried in ovens maintained at temperatures of approximately 240° F (yellow carroting). After drying, the skins may be stored for several months, or sent immediately to the brushing department. After treatment with the carroting solution, the fur is matted and irregular, and must be smoothed. This is done by brushers, who take each skin and subject it to a stiff revolving brush several times. From the brushing operation the treated skin goes to the cutting department where the fur is removed from the pelt by high-speed revolving blades which shear the fur and at the same time shred the pelt. The sheared fur then passes on to a conveying belt to the sorters, who remove rough scraps of skin cut by the machine from the fur. It is then packed in paper bags, each containing five pounds and is now ready for the hatting departments.

In some plants certain reclamation processes are conducted. Small scraps of fur recovered in the opening or sorting operations, or purchased from furriers, are chopped into small pieces and treated in one of two ways—the pieces may be glued to strips of manila paper and from then on handled as a pelt, or may receive vat treatment. After digestion in a vat, the scrap fur is freed of moisture in a centrifuge, then passed into a series of blowers which act as elutriators and free the fur of loose material. Other operations include storage and shipping of fur, and general maintenance and supervision.

Results of the Study

Preliminary Surveys.—Most of the thirty-six plants covered in the preliminary survey were in old frame buildings, many of them being establishments of the so-called "back shop" variety. Only a few were in brick buildings, and of the entire number only two could be considered well planned from the standpoint of control of any existing occupational hazards.

Although insanitary conditions may not be associated with ill health, it has long been recognized that the elimination of sources of uncleanliness in factories is conducive to the well-being and efficiency of the workers. For this reason, the general sanitary conditions of the thirty-six plants were observed, and were found, on the whole, rather poor. The data obtained showed that 57% were in either a poor or bad state of sanitation, 33% fair, and the remaining 10% good or excellent.

The data in Table 1 indicate the number of workers in each plant, and compare such figures with other industries in the United States. They show that there was a greater percentage of larger plants in the hatters' fur cutting industry than is the case in other industries. For example, 55% of the 210,959 plants covered in the 1930 United States Census employed less than ten persons, as contrasted with only 8.4% in the hatters' fur cutting establish-

ments. In fact, 20% of the thirty-six plants in the fur cutting industry employed 100 or more persons, as contrasted with only half that percentage for other plants in a typical industrial area. This may be due to the small number of plants in the fur cutting industry as compared with either the typical industrial area or the United States Census figures. However, it does show that most of these plants employed more than the average number of persons found in other trades.

TABLE 1.—COMPARISON OF PERCENTAGE DISTRIBUTION OF HATTERS' FUR CUTTING PLANTS WITH OTHER INDUSTRIAL ESTABLISHMENTS IN THE UNITED STATES ACCORDING TO NUMBER OF WORKERS

Type of establishment	Number of plants	PERCENTAGE OF PLANTS WITH LESS THAN:							More than 100 workers
		10 workers	20 workers	30 workers	40 workers	50 workers	75 workers	100 workers	
All industries in the U. S.*	210,959	55.0	72.0	77.0	81.2	85.2	88.8	91.9	8.1
Typical industrial area†	615	48.7	64.1	73.4	77.0	80.7	87.0	89.8	10.2
Hatters' fur cutting industry	36	8.4	27.8	38.9	52.8	63.9	69.5	80.6	19.4

* Personal communication from U. S. Census, 1930.
† *Public Health Bulletin No. 216.*

A comparison of industrial welfare provision in the hatters' fur cutting industry and plants in a typical industrial area is shown in Table 2. The number of plants covered in the industrial area was 615, as compared to 36 in the hatters' fur cutting trades. It is apparent that such facilities as medical care, first aid, etc., are rather limited in the latter, as compared with the plants in the typical industrial area. Only 5.5% of the employees in the fur cutting trades had the services of either a part-time or full-time physician, as compared to 15.3% in the typical industrial area. When one considers that even in the typical industrial area the welfare provisions are rather limited as judged by present standards, it is evident that the hatters' fur cutting industry has much to be desired in providing these facilities.

TABLE 2.—COMPARISON OF INDUSTRIAL WELFARE PROVISIONS BETWEEN HATTERS' FUR CUTTING INDUSTRY AND INDUSTRIES IN A TYPICAL INDUSTRIAL AREA

Industry	Number of employees	PERCENTAGE OF PERSONS WITH LISTED FACILITY				
		Physician	Nurse	First-aid room	Sickness records	Sick Benefit Association
Hatters' fur cutting	1,970	5.5	10.1	21.8	10.1	10.1
Typical industrial area*	28,686	15.3	34.1	48.5	40.0	29.4

* "The Potential Problems of Industrial Hygiene in a Typical Industrial Area in the U. S.," *Public Health Bulletin No. 216.*

The preliminary study disclosed that most of the employees had been in the industry ten years or longer, and a number for more than thirty years,

which is evidence that the industry is a stable one from the standpoint of labor turnover.

Sanitary and Occupational Surveys.—As a result of the preliminary study of the thirty-six plants, it was possible to select five factories for detailed investigation. The first step in an engineering study of this type is the making of a sanitary and occupational survey. Data pertaining to general sanitary conditions, ventilation, illumination, and environmental conditions, as well as air-borne toxic materials, were recorded; and information on the various occupations and the particular activities and raw materials associated with these occupations were noted.

In the hatters' fur cutting industry there were fourteen major occupations. Table 3 gives the number of workers by occupation in the thirty-six plants and contrasts these data with similar information on the five plants studied in detail. It may be seen that 57% of the workers were males, and that the

TABLE 3.—Occupational Analysis of the Hatters' Fur Cutting Industry Contrasted with Data Secured from Five Plants Studied

Occupation	Number of Employees in the Industry			Percentage of Total in:	
	Males	Females	Total	The industry	The five plants studied
(1)	(2)	(3)	(4)	(5)	(6)
Openers	24	133	157	7.9	5.5
Drummers and cleaners	57	0	57	2.8	2.0
Skin Sorters	13	11	24	1.2	0.9
Clippers	188	0	188	9.4	11.2
Carroters	180	2	182	9.1	7.3
Dryers and spreaders	52	45	97	4.8	7.0
Pilers	14	0	14	0.7	1.6
Brushers	74	26	100	5.0	4.4
Pasters and stickers	0	42	42	2.1	....
Cutters	145	2	147	7.3	8.6
Sorters	0	580	580	29.0	36.0
Packers	7	3	10	0.5	0.6
Shippers	22	0	22	1.1	2.0
General utility*	143	0	143	7.1	7.9
Blowers and choppers	135	0	135	6.7	5.0
Supervisors	60	8	68	3.4	....†
Clerical	23	15	38	1.9	....†
Grand Total	1,137	867	2,004	100.0	100.0

* Includes machinists, blade grinders, carpenters, elevator men, etc.
† Included in general utility.

occupation in which the greatest number of persons is employed is that of sorter—580 of the 2,004. A comparison of Columns (5) and (6), Table 3, gives an indication of the representative nature of the five plants selected for this study. In nearly every instance the percentage of workers by occupation in the five plants compared well with the figures for the industry as a whole.

Occupational Exposure to Mercury Vapor and Dust.—A summary of the occupational exposure of hatters' fur workers to mercury vapor and dust is presented in Table 4. Samples of mercury dust and mercury vapor were obtained simultaneously for each occupation and the results of the two separate analyses were added in order to determine the total exposure to mercury for each occupation.

A total of 124 samples of mercury vapor and dust were obtained for the study. Additional samples, taken to evaluate certain other factors (such as efficiency of ventilation and for a special study on the control of the mercury hazard), are omitted from this discussion. The occupations have been arranged in the order of magnitude of exposure to mercury vapor and dust, and it is quite obvious that the highest average exposure is for shippers, who are exposed to 7.2 mg of mercury per 10 cu m, and this high exposure may be attributed to the fact that they are employed in storerooms housing thousands of pounds of treated fur, which is constantly giving off mercury vapor.

TABLE 4.—Occupational Exposure of Hatters' Fur Workers to Mercury Vapor and Dust in Five Plants

Occupation	Number of samples	Number of workers	Average mercury exposure, in milligrams per 10 cu m	Occupation	Number of samples	Number of workers	Average mercury exposure, in milligrams per 10 cu m
Shippers	6	11	7.2	Clippers	9	61	1.5
Pilers	4	9	5.4	Chopped fur blowers	2	2	1.3
Blowers	13	24	4.6	Skin sorters	3	5	1.2
Cutters	13	47	4.0	White carrot dryers or spreaders	11	32	1.2
Sorters	26	197	3.8	Carroters	13	40	0.8
Blown fur packer	1	3	3.8	Openers	2	30	0.7
Brushers	8	24	3.1	Miscellaneous (office, machinists, etc.)	5	43	0.6
Fur chopper	1	2	2.8				
Drummers	4	11	2.5				
Yellow carrot dryers	3	6	2.0				
				Total	124	547	....

The next highest exposure was found for pilers, who are exposed to large quantities of fur skins treated with mercuric nitrate. These skins, after treatment, are subjected to a curing process consisting merely in allowing them to age over a period of several months.

The occupation of blower entails an exposure of 4.6 mg per 10 cu m due to the fact that the workers are subjected to the breathing of dust from fur which has previously been carroted. Cutting and sorting, which "go hand-in-hand," involve an exposure of 4.0 mg of mercury vapor, and dust, whereas the lowest exposure is associated with miscellaneous occupations, involving personnel in utility activities, who are exposed to 0.6 mg in 10 cu m of air.

In connection with the exposure to mercury vapor and dust, two interesting findings should be mentioned. The literature on mercurialism among hatters' fur workers is replete with statements concerning the severity of the hazard among carroters. The present study indicates that carroters are exposed to relatively small quantities of mercury, the findings as shown in Table 4 averaging only 0.8 mg in 10 cu m of air. A close study of the operations involved in carroting shows that these workers are subjected to the possible inhalation of mercury nitrate and nitric acid mist, generated during the application of the carroting solution to the fur with a hand or mechanical brush; and, apparently, the quantity of mist is not very great. In fact, mercury in the form of vapor or dust does not begin to appear until later processes, and in those workrooms where treated furs have been stored for a considerable time. The chemical

reaction which takes place subsequent to the application of mercury nitrate to raw fur, and which releases mercury vapor, is not definitely known. It is believed, however, that the reaction is somewhat as follows:

$$\text{Fur} + \text{Hg}\,(\text{NO}_3)_2 + \text{HNO}_3 \rightarrow \text{Keratin nitrate} + \text{HgO}\cdot\text{N}_2\text{O}_5$$
$$\text{Fur} + \text{O}\cdot\text{Hg}\cdot\text{N}_2\text{O}_5 + \text{H}_2\text{O} \rightarrow \text{Hg} + \text{Fur oxide} + 2\ \text{HNO}_3$$
$$\text{Fur} + \text{HNO}_3 \rightarrow \text{Keratin nitrate}$$

The reaction is slow, but it is well known that more mercury vapor is given off as the carroted fur becomes older. That there is some basis for these theoretical equations is indicated by the results for shippers, who are exposed to rather large quantities of mercury vapor emanating from the treated fur piled in the stockrooms.

It may be well to emphasize the second phenomenon observed. Following the preliminary survey made in the thirty-six plants, the first thought was that the small "back shop" type of plant with its extremely insanitary conditions would be the one in which considerable mercury exposure would be found; but actual quantitative studies indicated that these plants had relatively small quantities of mercury vapor and dust in the air, as compared with the larger plants. This may be explained by the fact that the small plants do not allow stock to accumulate, but work intermittently as orders are received. For this reason, little mercury is handled from day to day, and practically no treated furs are allowed to accumulate in the factories. On the other hand, in the larger plants, considerable quantities of mercury are being used, and hundreds of thousands of treated skins and vast quantities of treated fur are present continually. In addition, large quantities of treated and cut fur stock are allowed to stand in the various workrooms, often where there should be no mercury exposure, such as those in which preliminary processes are conducted, as in the room of openers and drummers. The remedy is apparent, since by segregation it would be possible to eliminate exposure to mercury for those workers whose tasks do not entail the handling of any mercury compounds or treated stock.

The exposure of workers to mercury compounds in the hatters' fur cutting industry was found to vary from a trace to a maximum of 10.4 mg per 10 cu m in the stockrooms. The exact effect of such exposures are discussed in another report dealing with the medical phases of this study.[17]

The present study afforded an opportunity to determine the effects of inhalation of organic dusts on workers, for which determinations of the dust concentration were made with the Konimeter device. This instrument was used because the nature of the dust involved precluded the use of the standard impinger method.

Table 5 presents the results of 112 determinations of the dust exposure. The highest was for drummers, namely, 16 million particles per cu ft. It is apparent that hatters' fur workers are exposed to relatively low dust concentrations, as judged by past studies of dusts associated with respiratory diseases.

[17] "A Study of Chronic Mercurialism in the Hatters' Fur Cutting Industry," by R. R. Sayers, P. A. Neal, R. R. Jones, J. J. Bloomfield, J. M. DallaValle, and T. I. Edwards, *Public Health Bulletin No. 234*, May, 1937.

Control Studies.—Three methods are used in the hatters' fur industry for the control of mercury vapor and fur dust—segregation, local exhaust ventilation, and general natural ventilation.

TABLE 5.—OCCUPATIONAL EXPOSURE TO DUST AMONG HATTERS' FUR WORKERS

Occupation	Number of samples	Millions of particles per cubic foot	Occupation	Number of samples	Millions of particles per cubic foot
Drummers	4	15.9	Brushers	16	4.2
Blowers	8	9.4	Carroters	13	4.0
Sorters	31	7.1	Clippers	6	3.1
Cutters	18	7.0	Spreaders	4	2.3
Openers	6	6.3	Shippers	4	1.8
Packers	2	4.5			
Total				112	....

Segregation.—Segregation is one of the most effective methods of control available. It has a definite application to those operators who are concerned with the handling of raw, untreated pelts—namely, openers, drummers, skin sorters, and clippers. Complete segregation of those employed in these occupations would free 21.3% of the workers from mercury exposure. Table 6 illustrates how exposure to mercury may be decreased by segregating drummers and clippers.

Local Exhaust Ventilation.—No data are available relative to the air movements required for the control of mercury vapor and fur dust by local exhaust systems. The efficiency of the air volumes used for the removal of mercury where exhaust equipment has been installed had never been studied. In fact, brushers, cutters, and blowers have been exhausted chiefly to eliminate the

TABLE 6.—EXPOSURE OF HATTERS' FUR WORKERS TO MERCURY VAPOR AND TREATED FUR DUST UNDER CONTROLLED AND UNCONTROLLED CONDITIONS

Occupation	Total Mercury Exposure, in Milligrams per 10 Cu M		Average air flow, in cubic feet per meter	Method of control
	Uncontrolled	Controlled		
Drummers	2.5	0.6	....	Segregation
Clippers	1.5	0.7	....	Segregation
Brushers	3.1	1.2	300	Local exhaust ventilation
Cutters	4.0	1.8	383	Local exhaust ventilation
Sorters*	3.8	1.7	383	Local exhaust ventilation
Blowers	4.6	0.7	2,000	Local exhaust ventilation
Pilers	5.4	Trace	....	Good natural ventilation
Storage workers and shippers	7.2	Trace	....	Good natural ventilation

* Depend on exhausted cutters.

nuisance caused by the dust generated, and even in these instances, efficiency has generally been based on the visual improvement secured. However, exhaust systems, as may be seen by reference to Table 6, decrease considerably the mercury exposure.

The method of control used on brushing machines is illustrated in Fig. 2. The exhaust opening is directly behind the revolving brush and the air is compelled to enter upward from the front of the machine which is open. An average of 300 cu ft of air per min per machine was found to produce the improvement indicated in Table 6. The reduction shown was obtained in a plant in which more brushes were operated than in the plants without control. This fact also implies the presence of more carroted skins in the brushing department which are continuously giving off mercury vapor.

Fig. 2.—Local Exhaust Ventilation for Brushing Machines

Cutting machines produce considerable dust due to vibration and to the fan action of the cutting blades, to which not only cutters, but also sorters, who are associated with them, are exposed. Thus (see Table 3), 36.3% of the workers in the industry are exposed to dust created by this operation alone. Moreover, these occupations are exposed to mercury vapor emitted by the large quantities of carroted skins and cut fur which are constantly present. The reduction through exhaust ventilation of mercury present in the air, both as vapor and as a constituent of the dust, is clearly shown in Table 6.

The manner of exhausting cutting machines is illustrated in Fig. 3. It will be noted that the hood is immediately behind the cutting blades, and draws air upward from the waste hopper below the machine. The sixteen cutting machines equipped with exhaust hoods each handled an average of 383 cu ft of air per min. The value was exceeded considerably on some machines close to the exhaust fans, but the increased air flows apparently did not interfere with the cutting operations.

Tables 5 and 6 indicate that blowers are associated with high mercury vapor and dust exposures, due to the presence of large quantities of treated fur and to the nature of the work done by blowing machines. To control the dust generated by one type of blower, an almost semicircular hood, concentric with the screened top of the blower, is used. The hood is constructed of varnished cloth in order to reduce the weight of the blower top, which must be removed periodically for cleaning. An adapter is fitted to the feeder end of the hood and connected to an exhaust leader. Air is thus compelled to move from the

Fig. 3.—Local Exhaust Ventilation for Cutting Machines

open end across the screened top of the blower and thence to the exhaust system. Ventilation studies made on this type of blower gave average air flows of approximately 2,000 cu ft per min.

General Natural Ventilation.—This method of control was found in piling rooms and fur-storage basements. Because the storage of large amounts of treated fur is productive of considerable quantities of mercury vapor, it is essential that adequate air changes be provided. Good natural ventilation was found to be effective in one piling room and one storage room, in which outdoor air was forced to sweep through the rooms for twelve hours each day. Natural ventilation must not be considered as adequate unless there is a continuous positive sweep of fresh air.

Recommendations

The recommendations which follow are based entirely on what has been found to be good engineering practice in the industry itself. However, it so

happens that the results of the medical studies suggest, for the present at least, a tentative limit of 2 mg of mercury in 10 cu m of air as a safe limit.[17] Reference to the results presented in Table 6 indicate that there are methods now in use in this industry which bring exposure below the threshold limit suggested by medical studies among the workers in this industry. Suggested remedies are as follows:

1. By segregating those operations handling untreated skins, approximately 23% of the workers may be removed entirely from the hazard of mercury exposure.

2. Local exhaust ventilation, properly designed and maintained for such operations as cutting, brushing, and blowing, will decrease the exposure for the workers in these occupations.

3. Good natural, or mechanical, ventilation decreased the quantity of mercury associated with the occupations of piling and shipping.

4. Since this study showed that in large plants considerable exposure may be attributed to the handling of mercury compounds in bulk, and to the storing of large quantities of treated fur, it is recommended that all treated material be removed from workrooms as quickly as possible and stored in well ventilated rooms.

5. Good "housekeeping" and general sanitation should serve to diminish the mercury concentrations in the workrooms. This study shows that mercury vapor is being generated constantly from treated fur. Hence, if treated fur skins and dust are allowed to accumulate, they will be a source of mercury vapor. It is recommended, therefore, that all floors, benches, and other objects on which dust may accumulate be swept and cleaned daily, either by wet methods or by vacuum. In addition, a complete general cleaning should be instituted once a week. In the larger plants it may be well to delegate the maintenance of protective equipment and the practice of good housekeeping to some responsible official who should make periodic inspections of all devices and methods used to minimize the mercury hazard.

6. As a result of this study it was shown that many of the plants have their processes very poorly arranged. Such practice is not only inefficient from the standpoint of production, but in this particular case serves to increase the exposure to mercury.

It is hoped that the methods outlined in the study of the mercury hazard in the hatters' fur cutting industry have served to indicate the rôle of the engineer in industrial sanitation studies. There are many other functions in studies of this nature, such as lighting, heating, safeguarding machinery, plant housekeeping, sewage disposal, water supply, cross connections, fire protection—in fact, those environmental conditions which form the basis of the work of the sanitary or public health engineer. It is not intended to minimize these latter activities because they have not been discussed in detail, but it is felt that such problems are fairly well understood.

Trends in Industrial Sanitation

The manifold functions of an engineer in the field of industrial sanitation, requiring as they do not only basic engineering training but also extensive

preparation in public health and allied subjects, may raise the question in the minds of many as to whether there is a potential field to make it desirable from both a professional and a financial standpoint to undergo the necessary training for this phase of public health. Perhaps a few statements concerning this matter may serve to throw light on the subject.

It is known that there are approximately 50,000,000 persons in gainful pursuits in the United States, all of whom may be said to come within the scope of an industrial hygiene program. Of this number 15,000,000 workers are employed in industrial occupations, many of which are associated with hazards capable of being dealt with by an industrial sanitation program. Numerous specific occupational diseases are also associated with the industrial environment which may give rise to excessive morbidity and mortality rates in the industrial population. Of equal, or possibly of more, importance than specific occupational diseases is the fact that the incidence of other diseases such as tuberculosis, pneumonia, other respiratory diseases, and degenerative conditions is very high in the industrial population. Attention has also been directed from time to time to the fact that the life expectancy of the industrial worker is less than that of the nonindustrial worker. All of these facts indicate that industrial hygiene is indeed an important public health function—one which is receiving more and more attention, not only by public health workers, but also on the part of industry and labor.

Further proof of the realization on the part of the public that industrial hygiene is a vital concern of the state is attested by the rapid growth of industrial legislation taking place in the United States. Where several years ago only a few states recognized the need for compensating workers injured by occupational diseases, today (1939) there are twenty-one states which do so. As a result of this trend in industrial legislation and the realization on the part of those charged with the administration of such laws that it is necessary to have proper machinery for preventing industrial health hazards, the few years since 1936 have seen rapid strides made in the establishment of industrial hygiene services in many of the states and large cities. This growth has been made possible in part through certain provisions of the Social Security Act. Although large industrial establishments are beginning to provide their own industrial health facilities, one must not lose sight of the fact that most of the plants in the United States are small. In fact, according to the 1930 Census 97.5% of the plants employ less than 500 workers, and of the nearly 9,000,000 workers in manufacturing establishments 61% are employed in plants of this size. It is apparent, therefore, that if industrial hygiene is to be brought to the bulk of the industrial population it will have to be accomplished by means of a governmental agency. As the result of these various factors, the expansion of industrial hygiene work has been quite rapid, and where in 1936 industrial hygiene was confined to one or two agencies of the federal government and several state departments of health and labor, today (1939) one finds that there are nearly thirty industrial hygiene units in various states and cities.

A few years ago those faced with the administration of industrial hygiene would have been appalled by the paucity of data available on the control of health hazards in industry. Although considerable work remains to be done

on this subject, sufficient data are now available concerning the effects of a large number of toxic materials and environmental conditions on the health of the worker, and what is more important, current knowledge of the methods for the control of many health hazards in industry has reached the stage where it may be applied successfully.

It should be apparent that all of these activities have resulted in a great demand for engineers capable of evaluating health hazards in industry and devising ways and means for their control. The burden of the problem resulting from the lack of trained personnel fell upon the U. S. Public Health Service for two reasons—its long experience in industrial hygiene work, and its administration of Social Security funds for this purpose. Realizing the urgency of this problem, the Division of Industrial Hygiene of the Health Service conducted two seminars of one month each for personnel selected to do industrial hygiene work in the state and city health departments. However, the demand for trained industrial hygiene workers still continues, and there is no reason to feel that this need will be satisfied for some time to come. Industrial establishments, insurance companies, universities, and state agencies are all calling for trained workers in this field of public health. It is apparent, therefore, that in view of the widespread interest in industrial hygiene, which in turn creates a demand for trained personnel, and because of the especial qualifications necessary to conduct industrial hygiene work successfully, adequate instruction should be available for those wishing to prepare themselves for a career in industrial hygiene. It is for these reasons that universities should give serious consideration to the institution of industrial hygiene courses as part of their regular or postgraduate curricula. Too much stress cannot be given to the necessity for such instruction in order that industrial hygiene workers may be prepared to deal with the problems concerning the health of workers in a manner that will produce substantial improvement and genuine progress in industrial life with the greatest efficiency and economy.

In order that an engineer may engage in industrial hygiene work he should be thoroughly trained in this field and should be well grounded in industrial processes. Such a person should have not only basic engineering training, but should be well versed, from a practical and theoretical standpoint, in the fields of microscopy, gas chemistry, physiology, and mechanics of ventilation, general sanitation, illumination, a knowledge of physical and chemical procedures, and industrial toxicology; and (most important of all), such an individual should have a broad public health background. It is apparent that a sanitary, mechanical, or chemical engineer, as such, does not fulfil these requirements exactly, but there is no reason why an individual with basic engineering training cannot, in time, prepare himself for work in industrial sanitation.

The duties and qualifications of an industrial hygiene engineer, which were adopted by the Committee on Industrial Hygiene of the State and Provincial Health Authorities, are as follows:

> "To determine under direction the necessity of making specific studies of particular industrial conditions; to conduct surveys and supervise studies

of factory conditions predisposing to occupational diseases; to prepare comprehensive reports of findings with recommendations for control of occupational disease hazards; to supervise the work of field and laboratory workers; and to do related work as required.

"The minimum qualifications call for graduation in chemical engineering, with 2 years' graduate work in industrial hygiene—to include ventilation, illumination, industrial toxicology, dust determinations; 3 years' experience in surveys and studies of industrial conditions for occupational disease control; or any equivalent combination of education and experience; familiarity with materials and processes used in industry; thorough knowledge of physical and chemical procedures for necessary determination of occupational disease hazards and of methods of control of these hazards; ability to recognize industrial processes and materials presenting potential occupational disease hazards; ability to enlist cooperation of plant executives, foremen, and laborers; initiative; tact; good judgment, and good address."

An attempt has been made to indicate the importance of the subject of industrial sanitation and the need today for trained workers in this field. The control of accidents and occupational diseases in industry has been shown to be largely an engineering problem, one which is varied and which has proved to be exceedingly interesting to those who have undertaken such work. It may occur to some that when many of the occupational diseases have been controlled the field for the engineer will become limited in scope and interest; but industrial sanitation, like many other endeavors, should continue to offer new opportunities and responsibilities. A field of industrial sanitation concerning which very little is known in the United States deals with the effects on health of recent trends in mass production. Little is known of the effect on the human system of work on the belt conveyor and the results of the "speed-up" in mass production industries. The subject of fatigue, which in all probability is associated with mass production, has been given little study. Still another subject in need of study is the effect of noise in industry on the health of workers. Another phase of industrial sanitation to which the engineer can make a definite contribution is that of housing, a subject that is receiving considerable attention. It is evident, therefore, that the field of industrial sanitation is not only virgin but will continue to be an interesting subject for many years to come—one which will test the ingenuity and ability of the engineering profession.

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PAPERS

PROBLEMS AND TRENDS IN ACTIVATED SLUDGE PRACTICE

BY ROBERT T. REGESTER,[1] M. AM. SOC. C. E.

SYNOPSIS

The activated sludge process is used to an increasing extent by cities in which local conditions require a high degree of sewage treatment. Problems presented in the design and operation of activated sludge plants in the United States, and trends indicative of American practice since 1930, are discussed in this paper. Pertinent data relating to the basic design and principal characteristics of thirty plants, of large and medium capacities, are included for comparative study. Available operation results for certain works are given. In conclusion, the need for further research is noted.

INTRODUCTION

The extensive use of the activated sludge process for municipal sewage treatment is a tribute to the efforts of early investigators in this field and to engineers who were responsible for its adoption on the first large-scale projects. Early experiments (1915 to 1921) demonstrated the high efficiency of the process in comparison with certain other methods and furnished basic information for the design of the large works at Milwaukee, Wis., Chicago, Ill., and Indianapolis, Ind. The operating results and experiences obtained from these and other works since 1925, together with further experimentation, have largely guided the design of subsequent works.

Advances in design and improvements in the methods of operation, particularly since 1930, have marked the endeavors to solve numerous pertinent problems. The general objectives have been: (*a*) More consistent performance under variable conditions; (*b*) greater economies in both construction and operation; (*c*) better technique for process control; and (*d*) increased facility of operation.

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **March 15, 1940.**

[1] Associate Engr., Whitman, Requardt & Smith, Baltimore, Md.

Activated Sludge Plants

The total nominal capacity of activated sludge plants in operation (1916 to 1938, inclusive) emphasizes the increased use of the process in the United States (see Fig. 1). The number of these plants exceeded two hundred in 1938. In 1939, with large works at Chicago (Southwest), and New York, N. Y. (Tallmans Island), included, the total nominal capacity in operation is estimated as 1,700 mgd.

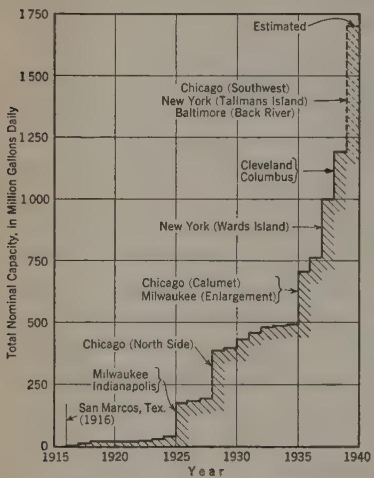

Fig. 1.—Increased Use of Activated Sludge Process of Sewage Treatment in the United States

The rapid increase of the total nominal capacity in operation during the period 1934 to 1938 reflects to some degree the stimulation from federal funds, although certain large projects constructed in this period were contemplated prior to federal participation. It is noteworthy that litigation involving the infringement of the process patents, extending from 1924 until the expiration of the last adjudicated patent on May 25, 1937, did not materially retard the adoption of the process for new plants.

Design data relating to thirty activated sludge plants of large and medium capacities are included in Table 1. Their total nominal capacity is approximately 1,600 mgd, and that of the ten largest works is nearly 1,400 mgd. Partial-aeration works, in series with trickling filters and other secondary treatment, at Cleveland, Ohio (Southerly), Decatur, Ill., Fort Worth, Tex., and elsewhere, are omitted from Table 1 so as to avoid possible misunderstanding of the data.

General Design.—With few exceptions, activated sludge plants of large and medium capacities, constructed since 1930, have included coarse screens, grit chambers, and settling tanks as preliminary treatment devices. These have been supplemented in some plants by pre-aeration chambers for grease separation. The devices for principal treatment have included the usual aeration tanks and final settling tanks. In a few instances, provision also was made for sludge re-aeration. Further treatment in the form of chlorination, filtration, and cascade or weir aeration has been provided in certain works. The maximum capacity for which the principal treatment devices are designed has shown a tendency to increase from 150% to 200% of the average sewage flow.

Based upon the experience at Chicago (North Side, Item 3, Table 1) of treating, satisfactorily, a sewage flow considerably in excess of the plant's design capacity, L. C. Whittemore, M. Am. Soc. C. E., has reported (1)[2] that the

[2] Numerals in parentheses refer to corresponding references in the Appendix.

hydraulic design for the channels and conduits in the Southwest works provide for a possible 25% increase in the designed maximum capacity of the tanks. This provision anticipates a possible reduction of the aeration period.

The general arrangement of the large plants in particular is characterized by tanks of greater unit sewage capacity—20 to 25 mgd in the case of aeration tanks, and 10 to 12.5 mgd for final settling tanks. This trend reflects a desire for construction economy. Compactness and flexibility are general features that have received careful consideration.

In some plants equipped with raw sewage pumps, these pumps are housed in the same building with the air blowers and auxiliary equipment and, occasionally, sludge de-watering equipment with incinerators is also included.

By-passing arrangements are often provided for the principal treatment devices. In the works at Columbus, Ohio (2), provision is made for pumping final effluent, during occasional periods of high water in the Scioto River. Under these conditions, it was intended that a portion of the sewage flow would receive principal treatment, while the remaining portion would be by-passed directly to the river after receiving preliminary treatment only.

Service tunnels containing miscellaneous piping and electrical conduits are a special feature of several works. These tunnels, by connecting the various structures, permit easy access for the maintenance of essential services and also provide sheltered communication.

Provisions for future extensions of plants allow for either the construction of additional similar units to be connected directly to the existing ones in accordance with the original plan, or for units of a different design to be located in unassigned adjacent space. In view of past changes in both processes and equipment, the latter method has merit.

The effort to obtain greater operating economies has influenced general design. The use of power derived from by-products has been an important factor in the selection of sludge disposal methods. A notable example is the use of waste heat from the incineration of flash-dried sludge, combined with the burning of coal, for the generation of steam to operate turbine-driven pumps, blowers, and generators at the Chicago (Southwest) plant (3). In contrast, the pumps and blowers in the New York (Tallmans Island) works are driven entirely by engines that utilize gas to be obtained from the digestion of sludge. This latter method is in use at Topeka, Kans. (East Side) (4*a*). In both cases, provision is made for the use of purchased gas when needed. Other plants use sludge gas in conjunction with purchased electric power for operating air blowers.

The decision to treat certain industrial wastes together with domestic sewage has been recognized as an important design consideration. In older plants (5), this was evidenced by relatively long aeration periods. At Greensboro, N. C. (6), where textile wastes are treated with the sewage, flexibility has been provided to permit the use of either the activated sludge process or other methods as may be found most suitable.

TABLE 1.—DESIGN DATA FOR THIRTY ACTIVATED SLUDGE PLANTS IN THE UNITED STATES

(Arranged in the Order of Their Nominal Capacities)

Item No.	Location	NOMINAL CAPACITY		YEAR OF STARTING		Type of sewerage system[i]	PRELIMINARY TREATMENT					AERATION		FINAL SEDIMENTATION, MIXED LIQUOR		Further treatment[cc]
		Sewage flow, in million gallons daily	Human population, in thousands	Construction	Operation		Coarse screen openings, in inches	Grit chambers	Period of preaeration for grease removal, in minutes	Number of mechanically cleaned settling tanks[p]	Sedimentation period, in hours	Mixed-liquor detention period, in hours	Return sludge, expressed as a percentage of average sewage flow[x]	Settling rate, in gallons per square foot daily	Detention period, in hours	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
1	Chicago, Ill. (Southwest)	400	1,300[d]	1935	1939	C[j]	2	No	11.0[b]	12	0.57	5.0	20	1,200	1.7	C[dd,ee]
2	New York, N. Y. (Wards Island)	180	1,230	1931	1937	C	1	Yes	No	8[q]	1.0	5.8	20	890	2.6	No
3	Chicago, Ill. (North Side)	175	800	1923	1928	C	1	Yes	No	8[q]	0.5	6.3	20	1,180	1.9	No
4	Milwaukee, Wis.	155	...	1920 1934[f]	1925 1935[f]	C	2	Yes	No	0[r]	...	5.7 and 6.0[f]	25	1,030[z] 1,080[f,aa]	2.3[f]	No
5	Chicago, Ill. (Calumet)	136	455	1931	1935	C	⅝	No	No	4	0.17	5.0	20[x]	1,200	1.8	No
6	Cleveland, Ohio (Easterly)	123	770	1931	1938	C	Yes[l]	Yes[n]	6.0	8[s]	1.0	5.0	25	980	2.2	C
7	Indianapolis, Ind.	75[b]	...	1921[g]	1925 1937	C[j]	1	Yes	No	4[t]	...	5.5[b]	30[b]	1,400[b]	...	No
8	Columbus, Ohio	50	400	1934	1938	C	¾	Yes	6.25	4[s]	1.5	6.0	30[v]	890	2.5	C[ff]
9	Baltimore, Md. (Back River)	40	...	1938[h]	...	S	¾	Yes[o]	No	5[q,u]	...	5.0	25	1,000	2.4	No
10	New York, N. Y. (Tallmans Island)	40	290	1937	1939	C	1	Yes	No	3	1.0	3.5	20	950	2.2	No
11	Providence, R. I.	36	...	1930	1936	C	¾	Yes[o]	No	3[q]	1.5	5.0	25[x]	870	2.3	No
12	San Antonio, Tex.	30	400	1929	1930	S	½	Yes	Yes	2[q]	0.5	5.0	20[x]	1,410	1.9	No
13	Peoria, Ill.	22[c]	140	1929	1931	C	1½	Yes	No	4[q]	1.5	10.5[w]	20	640[w]	4.2[w]	No
14	Houston, Tex. (North Side)	15[b]	...	1916 1929	1917	C	...	Yes	No	0	...	...	...[x]	...	...	No
15	Austin, Tex.	12	...	1936	1937	C	Yes	Yes[o]	...	1[u]	0.8	3.6	20	1,630	1.33	No
16	Phoenix, Ariz.	12	...	...	1932	S	1	No	No	1[q]	1.0	...	25[x]	770	2.7	C
17	Jackson, Mich.	11.25	...	1934	1937	C	Yes[m]	Yes	No	3	1.0	4.5	25	910	2.0	No
18	Pasadena, Calif.	11[b]	160[b]	1922	1924	S	15/16[m]	Yes	Yes	0[r]	...	5.5[b]	25[x]	700[b]	...	C

TABLE 1.—(*Continued*)

Item No.	Location	Nominal Capacity: Sewage flow, in million gallons daily	Nominal Capacity: Human population, in thousands	Year of Starting: Construction	Year of Starting: Operation	Type of sewerage system[i]	Preliminary Treatment: Coarse screen openings, in inches	Preliminary Treatment: Grit chambers	Preliminary Treatment: Period of preaeration for grease removal, in minutes	Preliminary Treatment: Number of mechanically cleaned settling tanks[p]	Preliminary Treatment: Sedimentation period, in hours	Aeration: Mixed-liquor detention period, in hours	Aeration: Return sludge, expressed as a percentage of average sewage flow[x]	Final Sedimentation, Mixed Liquor: Settling rate, in gallons per square foot daily	Final Sedimentation, Mixed Liquor: Detention period, in hours	Further treatment[cc]
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
19	Lansing, Mich.[a]	9	100	1937	...	C	Yes[m]	Yes	10.0	4[s]	1.0	6.0	25	1,120	1.5	No
20	Springfield, Ill.	7.5	75	1927	1928	C	¾, 1	Yes	8.0	4[v]	1.0	7.7	20	1,000	2.3	No
21	Lima, Ohio	7	70	1930	1932	C	¾	Yes[o]	No	2[q]	1.6	4.5	25	890	2.0	No
22	Greensboro, N. C.	6.5	40[e]	...	1938	C[k]	1[m]	Yes[o]	No	4	...	6.0	25[b]	920[b]	1.8[b]	C
23	Madison, Wis.	6.25	...	1934	1936	S	Yes	Yes	3.0	2	1.5	5.5	25	1,000	2.16	A
24	Charlotte, N. C. (Irwin Creek)	6	...	1927	1928	...[k]	⅜	No	No	0	...	5.0	...	...	1.5[bb]	No
25	Durham, N. C. (North Side)	6	...	1934	1935	S[k]	Yes[m]	No	No	6	...	5.0	...	...	...	No
26	Lancaster, Pa. (North)	6	29	1933	1934	C	¾	Yes	3.0	2	1.6	6.3	20	1,200	1.8	C
27	Lancaster, Pa. (South)	6	55	1933	1935	C[j]	¾	Yes	3.0	2	1.6	6.3	20	1,200	1.8	C
28	Topeka, Kans. (East Side)	6	...	1935	1937	C[j]	1	Yes[o]	19.5	2[q]	2.7	6.0	20[x]	820	2.4	No
29	Charlotte, N. C. (Sugar Creek)	5	...	1927	1928	...	2	Yes	No	1[q,r]	2.5	5.0	...	...	1.5[bb]	No
30	Ann Arbor, Mich.	4.5	48	...	1937	S	Yes[m]	No	12.0	2	1.0	6.0	25	880	2.25	No

[a] Garbage to be ground and digested. [b] Approximate. [c] Includes 8-mgd beer slop and strawboard wastes. [d] Plus 1,000,000 industrial equivalent. [e] Plus 20,000 industrial equivalent. [f] 70-mgd enlargement. [g] New units built in 1936. [h] Two each; aeration and final tanks not yet under construction. [i] C = combined; S = separate systems. [j] Packing house wastes in sewage. [k] Textile wastes in sewage. [l] Comminutors plus original 1½- and ¾-in. racks. [m] Comminutors. [n] Detritors plus original units. [o] Detritors. [p] Rectangular in shape, with the exceptions given. [q] Square. [r] Fine screens; are $\frac{3}{32}$ in. at Item 4. [s] Covered or housed. [t] Treat concentrate from fine screens. [u] Circular. [v] 3 rectangular and 1 square. [w] Based on 11-mgd sewage flow. [x] Sludge is re-aerated only at Items 11, 12, 14, 16, 18, and 28; provision for re-aeration has been made at Item 5. [y] One tank equipped for experimental use. [z] Maximum, 2,000 gal per sq ft daily. [aa] Maximum, 1,600 gal per sq ft daily. [bb] Sewage only. [cc] C = chlorination; A = cascade aeration. [dd] Partial flow used for maintenance of condensers. [ee] Part chlorination and part cascade aeration. [ff] Future chlorination.

Preliminary Treatment

Screening and Grit Removal.—Mechanically cleaned bar screens, or comminutors, are installed in most plants. The usual range in the width of clear openings for the screens is from $\frac{5}{8}$ in. to 1 in. Screenings are generally macerated and returned to the sewage. Usually, mechanically cleaned chambers are used for the removal of grit from combined sewage. An interesting exception is the omission of grit chambers at the Calumet plant and the omission of both bar screens and grit chambers at the Southwest plant in Chicago. In commenting (3) on this change, Mr. Whittemore and Norval E. Anderson, M. Am. Soc. C. E., have stated that these structures are costly to build and operate, and with the adoption of sludge incineration they believe that screens and grit chambers may not be necessary. However, the plants are designed to permit the future installation of grit chambers. Notes on the operation of the Calumet works (7) during 1937 indicate that no difficulty has been experienced in the removal of grit with primary sludge at times of normal sewage flow. On the other hand, periods of storm have shown the need for standby facilities to remove excess grit from the sludge.

Grease Separation and Removal.—A greater appreciation of the benefits to be obtained from the pre-aeration of screened sewage for the separation of grease is evidence by such provision in a number of plants with usual detention periods of 3 to 12 min. Grease is removed mainly with skimming devices in the preliminary settling tanks. The inhibiting effect of grease film on the rate of oxygen absorption by sludge floc stresses the importance of this step in pretreatment.

Preliminary Sedimentation.—It is interesting to note that, in nearly all of the thirty plants listed in Table 1, one or more preliminary settling tanks were provided, except where fine screens were installed. Doubts have been expressed as to the necessity for these tanks, and in at least one plant the tanks were by-passed for a period without apparent difficulty.

The principal reasons for including preliminary sedimentation in the Cleveland (Easterly) plant have been stated by George B. Gascoigne, M. Am. Soc. C. E., as follows (8): (1) To provide a sewage for aeration having uniform characteristics in that the effect of concentrated industrial wastes and of pollution from the first flush of storms would be minimized; (2) To provide optimum conditions for the rapid digestion of the deposited solids obtained from the excess activated sludge; (3) To reduce the volume of sludge that must be handled and disposed of; (4) To reduce the quantity of air required; and (5) To provide partial treatment for the first flush from storms.

Odor Control.—The location of activated sludge plants in close proximity to residential areas has necessitated means for the control of odors emanating from preliminary treatment devices. To this end, tanks at Cleveland and Columbus have been covered, and ventilation is provided by exhausting the air through a tall stack or, in the latter case, with incinerator flue gas. At Ann Arbor (9) and Lansing, Mich. (10), all of the preliminary treatment units are housed in a single building. Pre-chlorination of the sewage for odor control should be helpful for this problem.

Aeration

A marked trend is the tendency toward shorter aeration periods. At the Chicago (North Side) works (3), a battery of aeration tanks was operated without appreciable sacrifice in air economy or quality of effluent for nearly 2 yr with a detention period of between 3 hr and 3.5 hr. The weak character of Chicago's sewage was perhaps reflected in this performance. The design of the New York (Tallmans Island) plant (11) contemplates the use of only two aeration tanks to provide a detention period of 3.5 hr, and provision is made for the addition of a third tank if found necessary. The admission of settled sewage at three points in these tanks, after the introduction of return sludge, may assist in obtaining satisfactory performance. At Columbus, one aeration tank is equipped to operate in this manner for experimental purposes. The application of graduated aeration also tends toward a shorter period than 5 to 6 hr, as usually provided for treating domestic sewage.

Aeration Tanks.—The design of aeration tanks and their details has been described (12) by S. W. Freese, M. Am. Soc. C. E., who discussed many of the related problems and trends. In the arrangement of aeration tanks, a noticeable trend is the piping of one or a group of aeration tanks to a companion unit or group of final settling tanks. Although the idea was expressed in the design of the Charlotte (N. C.) plant (13), it has been expanded. At Charlotte, each aeration tank is constructed integrally with a final settling tank. At Durham, N. C. (North Side) (14), the tanks are similarly arranged except that an equalizing channel permits the use of various combinations when a tank is out of service. In the design of the plants at Baltimore, Md., and Columbus, where the aeration and final settling tanks are separate structures, aerated equalizing channels are provided with stop planks, so that companion units can be sectionalized for comparative operation under different conditions.

To eliminate the possibility of "short circuiting" in the flow channels of aeration tanks, and thereby prevent the leakage of under-aerated solids into the final tanks, occasional cross rows of diffuser plates are installed in several plants with the idea of furnishing agitation of the spiral core. At Baltimore, two vertical baffle walls are placed across each channel to serve the same purpose. The diffuser plates in the aeration tanks at Durham (North Side) are placed lengthwise in the center of each channel with the plates set in a vertical position. This arrangement produces two spirals of flow moving outward from the center at the surface.

Data relating to aeration tanks are given in Table 2. In the smaller activated sludge plants not included in the table, mechanical aeration devices of various types are preferred to diffused air, especially for nominal capacities less than 1,000,000 gal daily (15) (16). Of the works listed in Table 2, only Phoenix, Ariz., and Jackson, Mich. (Items 16 and 17), use paddle agitators combined with diffused air.

Graduated Aeration.—Graduated aeration (more popularly termed "tapered" aeration) appears to be a forward step in rationalizing operating procedure. The principle of "tapered" aeration is founded upon research which will be noted subsequently. It provides that the air applied to the sewage-sludge

mixture in an aeration tank shall be graded in accordance with the oxygen-demand curve, with the greatest rate applied near the inlet of a tank. The rate is decreased with the passage of flow through the tank. Probably the first installations to make large-scale use of this principle are at Providence, R. I.

TABLE 2.—DATA RELATING TO AERATION TANKS IN ACTIVATED SLUDGE PLANTS

Item No. (see Table 1)	Number: Total (1)	Number: Batteries (2)	Number: Average, per battery (3)	Design sewage flow[f] per tank, in million gallons daily (4)	Flow Channels: Number per tank[o] (5)	Flow Channels: Inside Dimensions, in Feet: Width, each (6)	Length, each (7)	Liquid depth over plates (8)	Total depth (9)	Diffuser Plates[u]: Rows per channel (10)	Number per tank (11)	Ratio of tank area to plate area[cc] (12)
1	16	2	8	25.0	4	32.75	434	15.0	17.0	2	2,820	20.2
2	16	4	4	11.25	4	22.25	345	15.0	17.0	2	2,240	13.7
3	36	3	12	4.86	2[p]	16.12[q]	420	15.0	17.0	2	1,440	9.4
4	24	2	12	3.54[g]	2	22.0	236	14.3	16.6	...	2,514	4.1
	12	1	12	5.83	2	21.5	370	14.6	16.6	2	1,316	12.1
5	22	2	11	6.19	1[p]	33.16[r]	425.5	15.0	17.0	1[v]	360 720[y]	39.0 19.5
6	16	4	4	7.69	2	27.0	334	14.5[t]	16.5	2[w]	1,315	13.7
7	10[a]	2	3 7	....	2[i] 4[s,a]	20.0[i] 30.0[a]	238	15.0	16	4[n] 3[a]	1,428[i]	13.3[i]
8	8	2	4	6.25	2	26.0	370	14.4	16.75	2[x]	1,328	14.5
9	4	2	2	10.0	2	30.25	383	15.0	17.33	2	1,232	18.8
10	2[b]	1	2	20.0	4	22.25	373	15.0	...	...	...	...
11	16	4	3 5	2.25	4[p]	15.0	115	10.0[t]	...	...[y]	...	...
12	6	2	3	5.0	3	20.0[s]	150	15.0[t]	16	3	...	14.3
13	4	1	4	5.5	3[p]	22.7	189	15.0[t]	...	...	...	...
14	5	..	..	....[g,h,j]	1[i,p]	18.0	280	9.75	...	...	...	7.0[t]
15	4[c]	1	4	3.0	1	...	...	...	...	...	...	...
16	5	1	5	2.4[j]	1[p]	27.0	330	14.0	...	1	260	34.3
17	4	1	4	2.81[j,k]	2	12.75	240	14.5[t]	16.5	1[z]	...	...
18	33[d]	..	..	0.24[l,g]	1[p]	10.0	67.5	15.0	...	...	...	...
19	4	1	4	2.25	2	15.0	210	15.0	16.5	2	...	...
20	5	1	5	1.5	2	16.0	169	14.0	...	2	...	...
21	4	1	4	1.75	2	15.0	189.5	9.5	11.5	2[aa]	504	11.3
22	4	2	2	1.63	1[p]	17.0	260	14.0	...	...	...	...
23	4	1	4	1.56	1[p]	30.0	135	15.0	17.0	3	336	12.0
24	6	1	6	1.0[m]	1[p]	15.0	200	11.0	...	3	504	6.0
25	6	1	6	1.0[m]	1[p]	...	...	...	...	2[bb]	...	...
26	3	1	3	2.0	2	...	...	...	...	...	...	11.6
27	3	1	3	2.0	2	...	...	...	...	...	...	11.6
28	2	1	2	3.0	3	20.0	139.5	15.0	...	3	...	10.9
29	5	1	5	1.0[m]	1[p]	15.0	200	11.0	...	3	504	6.0
30	4[e]	1	4	1.13	2	15.0	108	14.5[t]	...	2	...	...

[a] Three built in 1936. [b] Provision for third tank. [c] Two primary and two secondary. [d] Annular tanks, arranged about central final settling tank, added in 1938. [e] Each pair arranged to operate in series. [f] Spiral flow in each case except where noted. [g] Ridge and furrow. [h] Spiral flow in one tank. [i] Original tanks. [j] Spiral flow; paddles. [k] In second channel of each tank. [l] Baffles added and alternate air leads shut off to produce cross spiral flow. [m] Longitudinal furrow. [n] Double spiral flow. [o] Arranged in series except as noted otherwise. [p] In parallel. [q] One double width channel, 33.5 ft. [r] One double width tank 67.5 ft. [s] One double width channel, 40.8 ft. [t] Approximate. [u] Containers made of concrete in the case of Items 1, 2, 3, 4, 5, 8, 9, 10, 19, 21; aluminum, Items 6, 7 (new tanks), and 22; cast iron, aluminum lined, Items 24 and 29; and galvanized cast iron, Item 25; in Items 7, 18, 28 the tank floor is grooved. [v] Two rows in six tanks only. [w] Also five cross rows. [x] Also four cross rows. [y] Graduated aeration. [z] In first channel of each tank, 25% in second channel. [aa] Alternate plates in outer row. [bb] Diffusers arranged vertically in center of channel. [cc] Column (5) × Column (6) × Column (7) ÷ Column (11).

(17), and Jackson (18). In the Providence plant, the percentage of diffuser-plate area to tank-floor area is decreased in steps as the aeration period increases for flow through the tanks. In the Jackson plant, the first channel of the aeration tank contains a row of diffuser plates along the wall and the

second or return channel is equipped with paddle agitators on a central horizontal shaft. This latter channel also contains diffuser plates having an area of only one fourth of that in the first channel.

New equipment for the application of "tapered" aeration consists of a series of carbon tubes attached to an air manifold which can be swung upward out of the liquid in the tank for inspection and cleaning. Diffusers of this type are installed in the partial-aeration plant at Cleveland (Southerly).

Operating Galleries.—Operating galleries extending over the full width of a battery of aeration tanks, located between these tanks and the final settling tanks, are usually provided in the larger plants. In works where return sludge is mixed with settled sewage before entrance into the aeration tanks, mixed-liquor distributing channels from which each tank is fed are integral with the operating gallery. At Columbus and Baltimore, return sludge is introduced at the inlet end of each aeration tank, where it can mix with settled sewage that enters by a separate inlet. With this arrangement the sewage, sludge, and air are metered independently for each tank. The sewage meters are fed from a common channel or conduit integral with the gallery. The gallery also contains the air main, drains, return sludge pipes and all metering equipment.

A departure from the usual practice in providing superstructures for operating galleries has been made at Baltimore where metering instruments and hydraulically operated valve controls are housed in a centrally located return-sludge pumping station. The gallery on either side of this station does not have a superstructure. The metering instruments and valve controls are mounted on control tables of a type somewhat similar to those used in water-filtration plants. There are also instances in which partly housed operating galleries are used.

Flow Regulation.—In the plants previously cited, where sewage, sludge, and air are fed independently to each aeration tank, flow is regulated by the use of hydraulically operated valves inserted in the recovery tubes of the venturi sewage meters and in the individual air pipes. At Baltimore, sludge withdrawn from the final tanks, and sludge returned to the aeration tanks, is regulated by venturi rate-controllers of the type used with water filters, but with provision for flushing. Rate controllers have been used previously for sludge withdrawal at North Toronto, Canada (19). Control of the mixed-liquor flow from each aeration tank in the Cleveland (Easterly) plant (Item 6, Table 2) is regulated by a combination of venturi meter and cone-valve. This usage of regulating and metering equipment (20) is a trend toward the accurate proportioning of flows to the various units as desired.

Return Sludge Pumping.—Although air lifts are used at the Chicago (Southwest) plant (Item 1, Table 2), for economical reasons and because of the large quantity of return sludge to be handled (about 80 mgd), centrifugal pumps are more commonly installed.

To provide for the necessary variations in the rate of returning sludge, either variable-speed pumps or constant-speed pumps, operating against variable head, are necessary. Columbus has the first large plant to use the

latter method (Item 8, Table 2). Sludge from the final settling tanks enters a relatively small suction well in which the level fluctuates so as to balance the rate of pumping against the rate of inflow. This fluctuation increases or decreases the total pumping head, permitting the pumps to operate over a range of their combined characteristic curves, thereby varying the total rate of discharge. In order to keep the pumping head and the depth of the suction well to a minimum, pumps having flat characteristic curves are used. This method was adopted with certain modifications for the Baltimore plant (Item 9, Table 2).

Pumping facilities for return sludge in the large plants generally have a maximum capacity equal to 40% or 50% of the average sewage flow. This is exceeded in a few cases.

Blower Installations

Air blowers in use are of two general types—centrifugal and rotary positive-displacement. The latter type has been used with direct-connected gas-engine drive, which is favored by the relatively low speed of the blowers. Data relating to blower installations in thirty activated sludge plants are given in Table 3. In the ten largest works, the centrifugal type is used except at New York (Tallmans Island, Item 10, Table 3). Centrifugal blower units are driven either by steam turbines or by motors. Motors are of either the induction or synchronous type, the more expensive, latter type being used for power-factor correction. In some plants of medium capacity where rotary blowers are used, one unit is gas-engine driven and two units are motor-driven, or vice versa.

Flexibility for efficient operation over a wide range of total air capacity is often provided by having units of different capacities. In the case of centrifugal blowers of two sizes, the capacity of each smaller unit is usually from 50% to 75% of that of each larger unit. Since power cost for air blowing is a major part of the total operating cost of an activated sludge plant, increased attention has been directed to the selection of blower capacities. Centrifugal blowers, like centrifugal pumps, operate most efficiently at one point on the pressure-volume characteristic curve which is usually the rating point. The efficiency decreases very little within a limited range of intake volumes greater and less than the rated volume. Therefore, for a proposed installation it is necessary to select the sizes of blowers with proper regard for the probable air requirements—minimum, average, and maximum—to secure an economical arrangement.

Certain new features of centrifugal blower installations consist of: Independent structural supports for the blower units separated from adjacent parts of the blower building by elastic joint material; flexible joints in connecting pipes; cone-type check valves automatically operated; and blow-off valves for either the air discharge manifold or the discharge pipe of each blower. Separation of the blower supports from the building and flexible joints in the connecting pipes are provided as means of eliminating vibration in the building. Blow-off provisions are useful in minimizing the pumping effect when a blower is being added to those already in operation.

Motor-driven centrifugal blowers operate near a synchronous speed of 3,600 rpm and, therefore, require rather elaborate protective electrical interlocks.

TABLE 3.—DATA RELATING TO BLOWER INSTALLATIONS IN ACTIVATED SLUDGE PLANTS

Item No. (see Table 1)	Number of units	Installed Capacity, in Cubic Feet per Minute		Type of blower‡	Larger Units					Smaller Units					Ratio, capacity of smaller unit to larger unit
		Total	Per gallon of sewage		Number	Capacity of each, in cubic feet per minute	Rated discharge pressure, in pounds per square inch	Rated horsepower	Type of driver‖	Number	Capacity of each, in cubic feet per minute	Rated discharge pressure, in pounds per square inch	Rated horsepower	Type of driver‖	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1	3	180,000	0.65	C	3	60,000	7.75	...	S	None	...	...	...	...	...
2	6	227,000	1.82	C	4	42,500	7.75	...	M¶	2	28,500	7.75	...	M¶	0.67
3	7	250,000	2.06	C	4	40,000	7.75	2,160	M**	3	30,000	7.75	1,650	M**	0.75
4	6	220,000	2.04	C	2	50,000	10.0	...	S	4	30,000	10.0	...	S	0.60
5	4	140,000	1.48	C	2	40,000	7.75	1,750	M**	2	30,000	7.75	1,340	M**	0.75
6	5	170,000	2.0	C	3	40,000	7.4	...	M¶	2	25,000	7.4	...	M¶	0.63
7	4	91,000	1.75†	C	1	40,000	...	...	S	3	17,000	8.5	...	S	0.43
8	4	72,000	2.07	C	2	21,000	7.3	900	M¶	2	15,000	7.3	750	M¶	0.72
9	3*	49,500	1.78	C	3	16,500	7.5	...	M¶	None	...	...	...	...	...
10	4	60,000	2.16	R	2	20,000	8.0	800	G	2	10,000	8.0	450	G	0.5
11	3	37,500	1.50	R	3	12,500§	7.0	...	M**·††	None	...	...	...	...	...
12	4	36,000	1.73	C	4	9,000	8.5	440	M**	None	...	...	...	...	...
13	3	35,000	2.29	R, C	1	15,000	8.0	550	G	2	10,000	...	...	M	0.67
14	..	...	...	C	..	...	...	...	...	...	...	...	...	...	...
15	4	6,000	0.72	...	1	2,000	...	...	...	{2 {1	1,500 1,000	. . ...	... ...	... ...	... ...
16	4	4,000	0.48	C	4	1,000	...	...	M	None	...	...	...	...	...
17	3	12,000	1.54	C	2	4,500	7.2	250	M	1	3,000	7.2	150	M	0.67
18	5	20,000	2.62†	R	1	...	...	...	...	...	...	...	...	...	...
19	3	13,000	2.08	R	1	5,000	8.0	240	G	2	4,000	8.0	...	M	0.80
20	3	10,600	2.03	R	1	4,500	...	...	M	2	{3,900 {2,200	...	...	G	...
21	3	11,250	2.31	R	1	5,000	...	...	M**	2	{3,750 {2,500	...	...	M	...
22	3	5,250	1.16	R	3	1,750	7.0	...	...	None	...	...	...	...	...
23	2	7,800	1.80	R	..	4,500	8.75	230	G, M	1	3,300	...	...	G, M	0.73
24	3	9,000	2.16	C	3	3,000	8.0	170	M	None	...	...	...	...	...
25	3	4,500	1.08	C	3	1,500	6.0	60	M	None	...	...	...	...	...
26	2	8,800	2.11	R, C	2	4,400	...	...	M**	None	...	...	...	...	...
27	2	8,800	2.11	R, C	2	4,400	...	...	M**	None	...	...	...	...	...
28	2	8,100	1.94	R	2	4,050	8.5	180	G	None	...	...	...	...	...
29	3	9,500	2.74	C	1	3,500	8.0	225	G	2	3,000	8.0	150	M	0.86
30	3	6,500	2.08	R	1	3,000	8.0	120	G	...	{2,000 {1,500	... ...	100 60	M¶·†† ...	... ...

* Provision for fourth unit. † Approximate. ‡ C = centrifugal blower; and R = rotary blower, positive displacement type. § Low speed capacity, 6,250 cu ft per min. ‖ M = motor type; S = steam turbine; and G = gas engine. ¶ Induction type. ** Synchronous type. †† Two speeds.

In general, the design of large blower installations requires most careful attention if operating economy is to be realized.

FINAL SEDIMENTATION

Experience has shown that the final sedimentation step of the process deserves special attention if a clear effluent, free from floc, is to be secured. With this aim, refinement in the design of final settling tanks has been marked since 1930. A general lowering of settling rates, expressed in gallons per square foot of tank area daily, has resulted. As shown in Table 1, the customary design rate is between approximately 900 and 1,000 gal per sq ft daily on the average mixed-liquor basis. There are a few exceptions in which a rate of 1,200 gal per sq ft daily is used. On the maximum mixed-liquor basis, the settling rate is

between 1,600 and 1,800 gal per sq ft daily. Corresponding average detention periods range from 1.7 to 2.6 hr, depending upon the tank depths.

Final Settling Tanks.—Prior to 1930, final settling tanks were constructed square or octagonal in shape. Since that time, either circular or rectangular tanks have been built. The choice has often been the result of alternative bidding. Data relating to final settling tanks in thirty activated sludge

TABLE 4.—DATA RELATING TO FINAL SETTLING TANKS IN ACTIVATED SLUDGE PLANTS

Item No. (see table 1)	Number: Total	Number: Batteries	Number: Average per battery	Design sewage flow per tank, in million gallons daily	Shape‖	Influent: Point of inlet	Influent: Direction of flow	Inside Dimensions, in Feet: Diameter or width	Inside Dimensions, in Feet: Length	Inside Dimensions, in Feet: Side liquid depth	Effluent Channels in Each Tank: Type§§	Effluent Channels in Each Tank: Number	Effluent Channels in Each Tank: Total length of weir, in feet	Effluent Channels in Each Tank: Tank area per foot of weir, in square feet	Effluent rate per foot of weir, in thousand gallons daily
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1	32	2	16	12.50	C	Center	Radial	126	...	11.00	P	1	381	33	33
2	32	4	8	5.63	R	Center	Both ends	42.5	179	12.75	C	4	324	24	17
3	42*	3	14	...	{S	Side	Cross	77	77	12.75	C	3	443	13	..
					C*	Center	Radial	75	...	12.00	P	1	224	20	..
4	{11	2	5, 6	7.73	O	Two sides	Cross	98	98	14.00	C	3	588	15	13
	6	1	6	11.67	R¶	End††	Straight‡‡	84	161.5	13 82	C	7	1,176	12	10
5	15†	2	7, 8	9.06	S	Center	Radial	91	91	12.00	P	1	334	25	27
6	16	4	4	7.69	C	Center	Radial	112	...	12.00	P	1	339	29	23
7	26‡	2	12, 14	...	{O	....	....	42	78	...	..	...	...	..	..
					C‡	....	....	60	...	...	..	...	...	..	..
8	8	2	4	6.25	R	End	Straight	60	153	12.50	LC	2 each	672	14	9
9	4	2	2	10.00	C	Center	Radial	126	...	13.33	P	1	381	33	26
10	4	1	4	10.00	R	End	Straight	93.8	134	12.00	L	6	730	17	14
11	5	2	2, 3	7.20	S	Side	Cross	102	102	11.00	..	...	...	..	..
12	4	2	2	7.50	C	Center	Radial	90	...	15.00	P	1	238	27	32
13	4	1	4	5.50	S	Side	Cross	72	72	15.00	..	...	...	..	..
14	4	..	...	...	S	....	....	60	60	...	..	...	...	..	..
15	2	1	2	6.00	C	Center	Radial	75	...	12.00	P	1	...	..	..
16	1	1	1	12.00	S	....	....	140	140	11.50	B	1	350	56	34
17	4	1	4	2.81	C	Center	Radial	70	...	10.00	P	1	212	18	13
18	6§	..	...	1.33	S	Side	Cross	50	50	12.40	..	...	...	..	..
19	2	1	2	4.50	C	Center	Radial	80	...	9.00	P	1	...	..	..
20	4	1	4	1.88	R**	{....	Cross	60	60	12.80	P	1	...	..	..
						End	Straight	30	60	12.80	LC	2, 1	...	..	..
21	2	1	2	3.50	S	Side	Cross	70	70	9.87	S	1	...	..	..
22	2	1	2	3.25	C	Center	Radial	75	...	9.00	P	1	236	19	14
23	2	1	2	3.13	C	Center	Radial	70	...	12.00	P	1	220	18	14
24	6	1	6	1.00	R	End	Straight	15	52	10.80	E	1	15	52	67
25	6	1	6	1.00	R	End	Straight	15	50	11.75	C	1	...	..	..
26	2	1	2	3.00	R	End	Straight	44	68	12.00	LC	2, 1	180	17	17
27	2	1	2	3.00	R	End	Straight	44	68	12.00	LC	2, 1	180	17	17
28	2	1	2	3.00	C	Center	Radial	75	...	11.00	P	1	236	19	13
29	5	1	5	1.00	R	End	Straight	15	52	10.80	E	1	15	52	67
30	4	1	4	{0.75	R	End	Straight	16	65	11.00	LC	2 each	130	8	6
				1.50	R	End	Straight	33	65	11.00	LC	2 each	200	11	8

* Includes 12 tanks, added 1936–1937. † Space for one more. ‡ Twelve built in 1936. § Additional 78-ft diameter tank in 1938. ‖ C = circular; R = rectangular; S = square; and O = octagonal. ¶ Two revolving suction mechanisms each. ** 1 square, and 3 rectangular. †† Either or both. ‡‡ Or both ends to center. §§ P = peripheral; C = cross; L = longitudinal; B = bracketed launder; S = side; and E = end.

plants are given in Table 4. Circular tanks are often arranged in groups of four, fed from a common central well or by individual inlet pipes. On the other hand, rectangular tanks are arranged in batteries with either individual inlet

pipes or a common inlet channel. Common outlet channels are usually provided for both types.

Effluent Weirs.—For circular tanks, single peripheral weirs are in general use. Effluent weirs for rectangular tanks have been the subject of considerable variation in design. Four interesting arrangements of these weirs are shown in Fig. 2. Little information to indicate the best arrangement has come before the writer. In the tanks at Columbus (Item 8, Table 4), the weirs are adjustable so that any section, either longitudinal or crosswise, can be raised sufficiently to prevent overflow. Thus, it can be determined which combination of weirs will produce the best results.

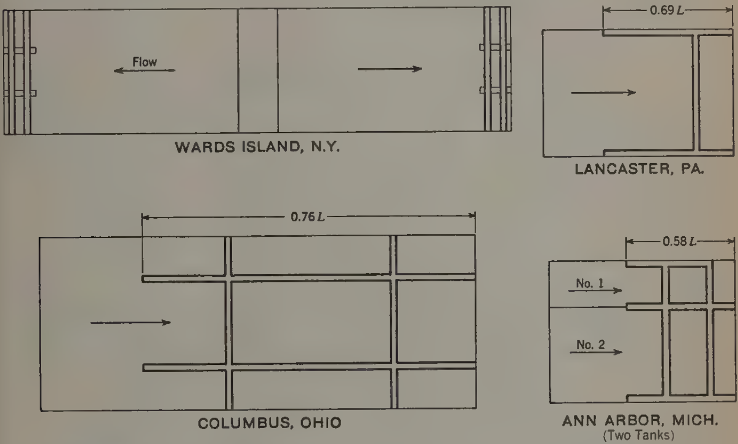

FIG. 2.—TYPICAL ARRANGEMENTS OF EFFLUENT WEIRS IN RECTANGULAR FINAL SETTLING TANKS

A comparison of total weir length, on the basis of tank area per linear foot, gives values from 8 to 24 sq ft in the case of the four plants included with Fig. 2, whereas for the largest circular tanks at Chicago (Southwest) and Baltimore (Items 1 and 9, Table 4) the value is 33 sq ft. However, the Chicago tanks have an overflow rate about 25% greater than those at Baltimore.

Sludge Level Control.—Several devices have been used to determine the depth of sludge blanket in final settling tanks. One of these is the photoelectric cell which was probably first tried at Morristown, N. J. (21). The possibilities have been appreciated and experimental work has been in progress at Chicago where it is expected that sludge-level control can be made largely automatic. At Baltimore and New York (Tollmans Island), the cells are used with indicating lights.

SLUDGE DISPOSAL

Probably no other branch of activated sludge practice has shown more divergent trends than methods of sludge disposal. The most common method in the smaller plants has been the return of waste sludge to the preliminary

settling tanks, with subsequent digestion of the combined sludge. In the larger plants, various other methods are used, such as barging to sea, heat-drying for fertilizer production, and incineration. Methods of waste sludge disposal in thirty activated sludge plants are noted in Table 5.

TABLE 5.—DATA RELATING TO THE DISPOSAL OF WASTE ACTIVATED SLUDGE

Item	Resettled in preliminary tanks	Concentrated	Digested with raw sludge	Dewatered		Dried or incinerated	Ultimate disposal
				On sand beds	By vacuum filters		
	(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	No	Yes*	No	No	Yes*	Flash-dried incinerated|| ||	Ash in fill
2	No	Yes†	No	No	No	No	To sea
3	No	No‡	...	...	...	......	......
4	No	No	No	No	Yes	Heat dried	Commercial fertilizer
5	No	Yes§	No	No	Yes	Flash-dried incinerated	Ash in fill
6	No	Yes||	Yes||	No	Yes||	Incinerated||	Ash in fill
7	No	No	Yes††	No	No	No	Farmers
8	No	Yes¶	Yes	No	Yes	Incinerated	Ash in fill
9	No	Yes**	Yes	No	Yes	No	Farmers
10	No	Yes**,†	Yes	No	No	No	To sea
11	Optional	No	No	No	Pressed	No	To sea
12	Part	No	Yes	Yes	No	No	Farmers
13	Yes	No	Yes	Yes	No	No	Farmers
14	No	No	No	No	Yes	Heat-dried	Commercial fertilizer
15	Yes	No	Yes	Yes	No	No	......
16	Optional	Yes**	Yes	Yes	No	No	Sold
17	Yes	No	Yes	Yes	No	No	Farmers
18	No	No	No	No	Yes	Dried	Commercial fertilizer
19	Optional	Yes¶	Yes‡‡	No	Yes	Optional	......
20	...	No	Yes	Yes	No	No	Farmers
21	...	No	Yes	Yes	No	No	Farmers
22	...	No	Yes	No	Yes§§	No	......
23	Optional	No	Yes	Yes	No	No	Sold
24	No	No	No	No	Yes	No	Farmers
25	Yes	No	Yes	Yes	No	No	Sold
26	Yes	No	Yes	Yes	No	No	Farmers
27	Yes	No	Yes	Yes	No	No	Farmers
28	Optional	Yes**	Yes	Yes	No	No	.....
29	No	No	Yes	Yes	No	No	Sold¶¶
30	Yes	No	Yes	No	Yes	No	Farmers

* Including some from North side. † Decanted in storage tanks. ‡ Pumped to Southwest and West Side. § In control tanks. || At Southerly. ¶ By centrifuges. ** "Picket-fence" thickening tanks. †† In lagoons or pits. ‡‡ Ground garbage to be digested with sludge. §§ Elutriation. || || With pulverized coal as boiler fuel. ¶¶ To private fertilizer company.

Concentration.—Separate concentration of the sludge prior to digestion is provided for in several large works. At the Cleveland (Southerly) plant (22), sludge from the Easterly plant is concentrated in Dortmund-type tanks. Circular sludge-thickening tanks of a different type, with vertical pickets attached to the arms of the revolving scraper mechanisms, are included in the New York (Tallmans Island) and Baltimore plants (Items 10 and 9, Table 5). These thickeners are used at Topeka (4*b*) and at Phoenix (23) (Items 28 and 16, Table 5). The method was first tried at the Los Angeles (Calif.) Experimental Station (24). It was found desirable to apply chlorine near the surfaces of these tanks to delay septic action, and chlorine installations are provided in the plants mentioned.

Centrifuges of a modified high-speed "cream separator" type are installed at Columbus (five units) and at Lansing (one unit) for the concentration of waste sludge. Each machine has a capacity of about 1,500 gal per hr. In the case of the concentration tanks the decanted supernatant liquor is discharged with the plant effluent whereas in the case of the centrifuges the supernatant liquor is returned to the preliminary tanks.

Digestion.—Digestion with subsequent dewatering of a mixture of raw and waste activated sludge is the most common method of sludge disposal. In plants of medium capacity, it is attractive from the standpoint of effecting operating economies through the use of gas, and in producing a reduced quantity of inoffensive sludge suitable for land use after dewatering on sand beds or by vacuum filters.

Heat-Drying.—The heat-drying of activated sludge in rotary driers to prepare it as commercial fertilizer has been confined principally to the large installation at Milwaukee (Item 4, Table 5) and to the smaller plants at Pasadena, Calif. (Item 18, Table 5), and Houston, Tex. (North Side) (25) (Item 14, Table 5). For a long time, the odor problem from drier gases was acute at Pasadena (26), but subsequently was solved. Prior to heat-drying the sludge is dewatered by vacuum filters.

Incineration.—The Chicago (Southwest) (Item 1, Table 5) and Calumet (27) (Item 5, Table 5) plants have notable installations for the incineration of undigested waste sludge after dewatering and "flash"-drying. If the dried sludge can be sold reasonably it may be utilized for fertilizer. On the other hand, the Cleveland (Southerly) and Columbus (28) (Item 8, Table 5) plants include the incineration of digested combined sludge after dewatering. Again, vacuum filters are used for dewatering.

Further Treatment

An innovation in the further treatment of the effluent from activated sludge works is weir or cascade aeration for the introduction of additional dissolved oxygen to the effluent. This possibility was observed at Chicago, and a chamber for the purpose is installed at the Southwest plant. An interesting type of cascade aerator, to receive the effluent from a concentric magnetite filter, is provided in the Ley Creek Plant near Syracuse, N. Y. (29). Consideration is being given to the use of magnetite filters for the further reduction (30) of the suspended solid content and the biochemical oxygen demand of activated sludge effluent. These filters have been installed already in several plants where the local conditions have warranted their use. Where seasonal treatment is practiced, by means of double sedimentation without aeration, subsequent filtration of this type offers the means for obtaining an improved effluent.

Operation and Control of Process

The operation of activated sludge works and the control of the process have presented a variety of problems. Over a period of more than fifteen years, experience and knowledge have gradually accumulated from the operation of large works. As the result, an approach has been made to the standardization

of operating procedure and technique. However, much work remains to be done in this regard.

In general, the results obtained from the use of this process have been satisfactory and have suited the requirements of various localities (7) (9) (18) (31) to (36). Average annual operation data for nine representative activated sludge works are given in Table 6. In these plants, activated sludge operation is continuous through the year. Certain other works are operated on a seasonal

TABLE 6.—AVERAGE ANNUAL OPERATION

Item No. (see Table 1)	Location	Year	Sewage Flow: Million gallons daily	Sewage Flow: Ratio to design flow	Parts per Million — Suspended Solids: Raw sewage	Suspended Solids: Final effluent	Biochemical Oxygen Demand[f]: Raw sewage	Biochemical Oxygen Demand[f]: Final effluent
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
3	Chicago, Ill. (North Side)	1937	204.8	1.17	140	12	107	9.2
4	Milwaukee, Wis.	1937	116.3					
	Original works		75.4	0.89	278[d]	18	166	8.8
	Extension		40.8	0.58	278[d]	15	166	6.6
5	Chicago, Ill. (Calumet)	1937	65.4		151[e]	15	95	13.0
7	Indianapolis, Ind.	1937	49.2	0.76[c]				
	Activated sludge		25.3	...	323	14	215	17
	Plain aeration		23.5	...	323	...	215	89
12	San Antonio, Tex.	1937	16.8	0.56	316	44	190	28
16	Phoenix, Ariz.	1935	9.94	0.83	177	23.4	170	23.1
17	Jackson, Mich.	1937[a] 1938[a]	7.01	0.62	178	15	115	2.7
20	Springfield, Ill.	1937	8.7	1.16	191	12	151	13
28	Topeka, Kan. (East Side)	1937[b] 1938[b]	5.6	0.93	322	23	302	26

[a] Six months of each year. [b] Eleven months. [c] Approximate. [d] Screened sewage. [e] Iron as Fe, 11 ppm. aeration 8.75 hr. [h] In aeration tanks only. [f] Effluent.

basis with aeration omitted during months when relatively high flows prevail in the receiving streams. J. R. Collier, of Elyria, Ohio, and E. E. Smith, Assoc. M. Am. Soc. C E., have both reported (37) that their respective plants were operated with seasonal aeration, and that by careful control the annual starting of the process presented no abnormal difficulties. Of course, operating economies in power consumption are achieved by seasonal treatment.

Starting Procedure.—During the period of initial operation and adjustment of an activated sludge plant, many unexpected problems arise which test the skill of the operating force. Valuable lessons are learned which contribute to the advancement of design. Interesting experiences of starting operation at Ann Arbor (9), Lancaster, Pa. (two plants) (38), Peoria, Ill. (39), and elsewhere, have been reported.

The concensus of opinion of Ohio operators (37) regarding the preferred method for producing activated sludge (undoubtedly for small plants) is briefly, as follows: (1) To fill the aeration tank with freshly settled sewage and to aerate for a period; (2) to settle this aerated sewage and decant the clear portion to the final settling tank; (3) to replace the decanted liquor with settled

sewage, and to repeat the operation until enough activated sludge has been produced to satisfy the requirements of incoming sewage.

About one week is usually required to establish the process and to develop satisfactory activated sludge. At the Lancaster (North) plant, three weeks were required during cold weather.

Control of Process.—Control of the process has shown a distinct trend toward simplification. Valuable criteria of performance are the suspended

DATA FOR ACTIVATED SLUDGE PLANTS

Detention period in aeration tanks, in hours	Return Sludge (Percentages)		Percentage of suspended solids in mixed liquor	Air, in cubic feet per gallon of sewage	Settling rate of mixed liquor, in gallons per square foot daily	Final Effluent (Parts Per Million)		Over-All Removal (Percentages)	
	Sewage flow (10)	Solids (11)				Dissolved oxygen (15)	Nitrates (16)	Suspended solids (17)	Biochemical oxygen demand[f] (18)
4.2	22.5	1.04	0.23	0.35	1,290	2.0[m]	2.66	91.5	91.5
6.0	32.3	1.61	0.40	1.26[h]	978	...	4.5	93.5	94.8
9.74	29.3	1.76	0.41	1.21[h]	666	...	4.5	94.5	96.0
5.3	22.5	1.48	0.269	0.38	1,063	7.3[n]	4.43	90.1	86.3
10.46	27	1.04	0.26	1.25	617[l]	...	...	95.6	92.1
9.94	None	...	...	0.41	894[l]	...	...	...	58.5
7.56	44.4[i]	...	...	1.00	689[l]	1.7	1.2	86.0	85.5
5.7	29.3[j]	0.88	0.167	0.30	632[l]	2.3	2.13	86.6	86.4
...	15	2.2	0.332	0.8	...	6.7	10.1	92	98
6.4	23	0.87	0.189	0.58	1,200	...		93.4	91.4
5.3 to 6.9[g,h]	27.3 to 39.4[h]	0.21 to 0.44[h]	0.08 to 0.156[h]	1.14 to 1.53[h]	...	1.0 to 2.9[h]	...	93	91

[f] Five-day at 20° C. [g] Re-aeration 3.9 to 4.9 hr. [h] Monthly averages. [i] Re-aeration 1.12 hr. [j] Re-
[m] At outfall sewer, 8.9 ppm. [n] At outfall sewer.

solids in the mixed liquor, the dissolved oxygen in the liquid of both the aeration and final settling tanks, and the sludge index and microscopic examination as warnings of the possible "bulking" of the sludge. In the application of these criteria, the tendency has been to reduce the time element so that corrective steps can be taken when needed. The technique has been simplified (40) (41) so that any intelligent shift operator can perform the necessary tests.

Suspended Solids in Mixed Liquor.—The optimum suspended solid content of the mixed liquor in the aeration tanks is best determined by actual plant performance. It is related to the character of the sewage to be treated, the quantity of air introduced for the maintenance of biological activity, the detention period, and possibly other factors. The rate of returned sludge required to maintain a suspended solid content will vary, of course, with the percentage of solids in the returned sludge. This relationship is shown by the curves in Fig. 3, with the average suspended solid content of the mixed liquor superimposed for each of seven plants (the curves neglect the suspended solids in the settled sewage). The wide variation in the practice of maintaining solids in the aerated mixture is evident and apparently stresses the influence of local

operating conditions. In this regard, the 1937 report on sewage disposal at Indianapolis (36) states that mixed-liquor solids were held at approximately 0.25% and that over-aeration, as shown by turbid effluent and foaming in the aeration tanks, resulted when the solids reached 0.30%.

In a letter dated December 23, 1938, Wellington Donaldson, M. Am. Soc. C. E., states that the suspended solids in the New York (Wards Island) aeration tanks were reduced from 1,500 ppm to 1,300 ppm (0.13%) in the belief that the lower value would permit both air economy and a larger margin for sludge storage when necessary.

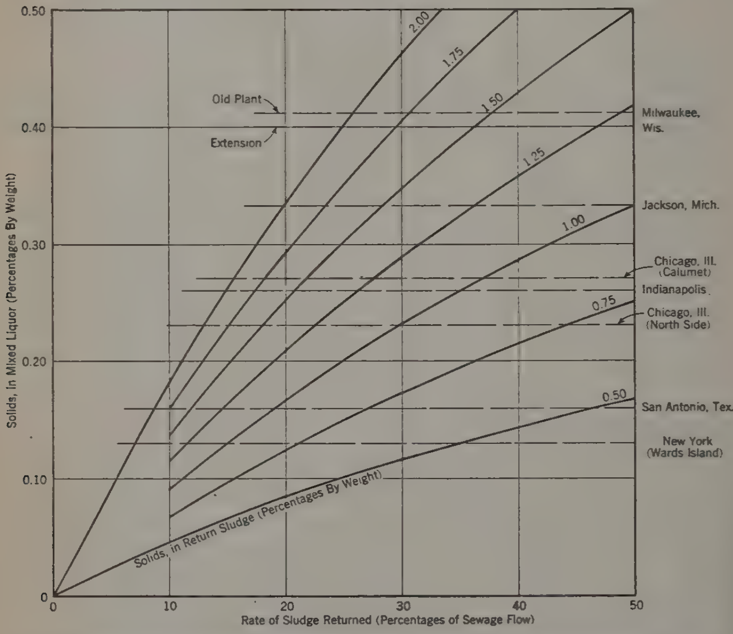

FIG. 3.—RELATION OF SOLIDS IN RETURN SLUDGE AND IN MIXED LIQUOR TO THE RATE OF SLUDGE RETURNED (1937 PRACTICE OF MAINTAINING SOLIDS IN AERATED MIXTURE IS SUPERIMPOSED)

The laboratory centrifuge is used in numerous plants to determine the percentage of suspended solids rapidly. The values obtained from the calibrated centrifugal readings are later checked against gravimetric determinations.

Dissolved Oxygen.—The maintenance of dissolved oxygen in the effluents of both the aeration and final settling tanks, to the extent of at least 2 ppm, is recognized as a necessity for proper operation. In some plants a higher value has been found desirable. In notes dated December 22, 1938, L. M. Johnson, engineer of maintenance and operation, the Sanitary District of

Chicago, with reference to the dissolved oxygen test, states that it is their practice to collect samples in wide-mouthed quart bottles to which about 2 cu cm of a 10% copper-sulfate solution has previously been added. Immediately, the copper sulfate retards the oxygen demand of the sludge in the sample, and also coagulates it for rapid settling. As soon as the sludge has settled sufficiently, the supernatant liquor is siphoned into an 8-oz bottle and treated with the Winkler reagents. Instead of titrating, these bottles are compared colorimetrically with known standards. Mr. Johnson's experience has indicated that dissolved oxygen determinations should be made every 4 hr. Samples are collected at the discharge end of one of the aeration tanks in each battery, as well as at various stages down one of the aeration tanks.

A helpful method of control used for the routine operation of the Indianapolis works was described by Don E. Bloodgood, Assoc. M. Am. Soc. C. E. (42) (43). A machine for obtaining the oxygen demand and sludge activity is used daily to check the performance of the plant. Another machine to measure the rate of oxygen utilization (44) has been used as a control device.

Sludge Index.—The sludge index, or the ratio of percentage of sludge by volume, after settling for 30 min, to the ratio of suspended solids by weight, in the mixed liquor, is used principally as a means of detecting the probability of "bulking" sludge. This test is widely used, but in some instances the period of settling is different.

At the Chicago (North Side) plant, where the 30-min period is used, a normal sludge has an index of less than 75, whereas a poor-settling sludge will have an index of several hundred. It has been found that the index correlates closely and varies inversely with the ash content of the sludge. A steady increase of the sludge index denotes "bulking" and suggests that remedial measures be adopted.

Daily microscopic examinations of the sludge for the presence of Sphaerotilus growth is another means of forecasting "bulking" troubles. Types of "bulking" and the various remedies tried are discussed fully in a committee report of the American Public Health Association (45).

Sludge Blanket.—The level of sludge, or the depth of sludge blanket maintained in the final settling tanks, may have a detrimental effect upon the quality of the sludge unless the level and quantity stored is kept to a minimum consistent with plant rquirements (37). A lowering of the dissolved oxygen in the final tanks from any cause is likely to destroy the biological equilibrium and result in "bulking" if a deep blanket is allowed. A high sludge level may also cause passage of floc with the effluent. For these reasons, devices to indicate or to control the sludge level should be useful.

The density of sludge, as previously stated, is related to the rate of returned sludge required to maintain a pre-determined, suspended-solid content in the mixed liquor (Fig. 3). It is probable that increased depth of sludge blanket, with consequent compaction, produces a greater density of the sludge. Therefore, relatively deep draw-off hoppers of limited capacity and continuous stirring may be advantageous. To attain this end, the circular final settling tanks at Baltimore have a steeper bottom slope near the center than near the periphery.

The stirring mechanism, within the limits of the center cone, is equipped with "pickets" for thickening.

Disturbing Influences.—It is well known that certain influences have a harmful effect upon the satisfactory performance of activated sludge works. Some of these factors are septic raw sewage, variations in the quantity and strength of the incoming sewage, the return of strong supernatant liquid from digestion tanks to the sewage ahead of preliminary sedimentation, and relatively large proportions of certain industrial wastes in the sewage. The effect of septic sewage may be reduced by pre-chlorination in the outfall sewer upstream from the plant.

The production of a higher quality supernatant liquid by the use of two-stage digestion should aid to some extent in overcoming difficulties from this source. In the New York (Tallmans Island) plant, provision is made for a 6-hr detention of the supernatant liquid prior to its introduction to the plant influent (11).

With regard to industrial wastes the process has been used succesfully for treating sewage containing appreciable quantities of wastes from packing houses, vegetable canneries (43), rayon mills (46), and other sources. On the other hand, wastes from metal works, dairies, textile mills, and diverse industries have proved troublesome, and in some cases such known wastes may have precluded the use of the process. Noteworthy research on the treatment of sulfur dye wastes with sewage by the activated sludge process was undertaken by the Textile Foundation on a semi-plant scale (47) and demonstrated the successful use of the process.

Economies.—The practice, since 1930, of reducing the quantity of air to effect operating economies and thereby decrease the nitrate content of activated sludge effluents was discussed (48) by F. W. Mohlman. He noted that in cases where additional downstream dilution does not occur for a long period, nitrification of ammonia contained in the plant effluent may result in the disappearance of dissolved oxygen in the stream to the detriment of fish life. The retardation of luxuriant growths of algae is perhaps another benefit derived from the maintenance of a lower content of nitrate in the effluent.

Fundamental Research

The perplexing character of many problems encountered in practice, together with the desire for a more complete understanding of the fundamental phenomena and the environmental factors of the activated sludge process, has stimulated research in this field.

Important findings of the numerous studies undertaken by the Division of Stream Pollution Investigations, U. S. Public Health Service ((49) to (52), inclusive), the Sanitary District of Chicago (53), the Division of Water and Sewage Research, New Jersey Agricultural Experiment Station, and other research groups (44) (54) (55) were reported from 1930 to 1938. Although the complete theory of the process has not been established, the studies just mentioned have contributed greatly to this end.

Needed Study and Research

Because of the comprehensive scope of the subject and the wealth of related data, this paper has been necessarily confined to a general discussion of the problems and trends in activated sludge practice. Detailed consideration of specific problems has been omitted. The need for additional study and research is evident. Certain phases for further investigation, leading toward the standardization of practice, are noted as follows:

(1) The influence of basic factors such as the character of sewage, variation in flow, and the degree of pre-treatment upon plant performance, the object being to determine design limits.

(2) The effects of certain industrial wastes and sewage upon normal functioning of the activated sludge process, and proper means for the adjustment of the process or for partial treatment of the wastes at their source.

(3) Desirable methods for the treatment of the supernatant liquid from digestion tanks prior to introduction of the liquid to activated sludge treatment devices. Determine the best point of application.

(4) Comparison of several methods for thickening activated sludge prior to digestion, and the formulation of desirable operating procedure.

(5) Starting procedure for the process with recommendations as to suitable practice particularly for small plants.

(6) Parallel comparison of graduated aeration and uniform aeration, to determine relative advantages, under similar conditions of routine operation.

(7) The possible general relation between the optimum suspended-solid content of aerated mixtures and the character of settled sewage.

(8) The annual increase in air-pressure loss for diffuser devices. The recovery of pressure loss following the cleaning of these devices, the results preferably expressed as percentages of the original loss. Tests to be performed under normal conditions of continuous operation.

(9) From the preceding study, to establish both the economical periods of cleaning and useful service for diffuser devices of common porosities, consideration being given to the balancing of costs for cleaning, replacement, and power consumption. These data should be helpful in selecting the economical rating pressure for centrifugal blower installations.

(10) Extension of early studies on measured air losses for pipes of various sizes and different materials to obtain suitable design coefficients for use with the Fritzsche formula. Measurement of the air-pressure loss in fittings and valves of various types.

(11) The minimum quantity of free air required for the satisfactory agitation of aerated channels having various depths, with results expressed in terms of cubic feet per minute per square foot of channel surface.

(12) The efficiency of final settling tanks and factors affecting their design. Consideration of inlets and overflow weir arrangements.

(13) The relation of depth of sludge blanket to the density of activated sludge in final settling tanks.

(14) The rate of absorption of oxygen by effluents when subjected to cascade or weir aeration with various heights of fall, degrees of agitation, or surface area exposed.

(15) Improvements to the methods and technique of process control through a continuation of research relating to the fundamental phenomena and theories involved in the process.

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By John F. Curtin,[1] Jun. Am. Soc. C. E.

Synopsis

Between 25% and 50% of the total cost of vehicular bridges and tunnels is expended on their approaches; and yet relatively little of the diligent research and analysis which accompanies this expenditure has been made available to the engineering profession. In this paper an attempt is made to review the more significant elements of bridge and tunnel approaches and to make suggestions for their design. The scope of inquiry comprehends three types of approaches: Direct extension, reservoir, and tapered plaza with feeder connections. These are reviewed summarily, with a brief comparison of their relative merits and disadvantages.

The development of approach connections is outlined from the requirements of traffic and the adequacy of existing routes. In addition, the requirements for design speed, sight distance, curvature, and other alinement features of approaches are reviewed. The design of bridge and tunnel plazas is considered, based upon their functions of toll collection and lane convergence, and a discussion of decentralized plazas, reservoir space, layout of plazas, and portal transitions is included.

In this paper particular attention is given to the toll structure because of the increasing use of the toll type of river crossing and the more stringent requirements which are made of it. The kinetics of vehicles is the paramount consideration throughout, for it is believed that successful operation is dependent entirely upon smooth, uninterrupted traffic flow.

Definitions

Bridge (and tunnel) approaches have been defined by the late Wilbur J. Watson, M. Am. Soc. C. E., as follows:[2]

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by March 15, 1940.

[1] Asst. Engr., Pennsylvania Turnpike Comm., Harrisburg, Pa.

[2] "George Washington Bridge: Approaches and Highway Connections," by J. C. Evans, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 97 (1933), p. 436.

"The approaches to a bridge comprise the traffic arteries leading to the ends of the bridge proper, and such adjustments of alignments and grades of said arteries in the immediate vicinity of such ends as is necessary to afford the maximum convenience of access, and render available to the public the entire capacity of the bridge proper."

A considerable variety of approaches has been developed to secure the interchange of traffic which this definition implies, but, for review purposes, they may be classified into three broad groups as follows: Direct street or highway extension, reservoir, and tapered plaza with feeder connections. These types are not sharply defined, the difference being mainly in principle of operation. The approaches to many existing river crossings embody the characteristics of more than one group and some fall beyond these classifications entirely, but for the most part, the methods of operation outlined herein are applicable to most terminal facilities.

Direct Street or Highway Extension.—Because of its simplicity, the direct street or highway extension is still the most commonly constructed type of bridge or tunnel approach. Thousands of bridges are in use throughout the United States on which the connecting roads lead directly to the structure proper. In most cases, the river crossing is regarded as an integral part of a particular traffic route, rather than one which is to link several routes at each end, and for this reason the design requirements on its approaches are the same as those on the route which it connects. Little planning beyond a consideration of the maximum gradients in bringing the bridge or tunnel roadway to grade is required. Whether tolls are collected or not makes little difference, for the volume of traffic seldom requires more than a slight widening of the roadway through the toll collection area in order to provide space for the booths.

This type of approach has been well applied in rural areas, even to the ends of large structures, one example being the Marin County approach to the Golden Gate Bridge in California. Where two or more highways converge toward a river crossing, the direct extension has been modified in some cases with a traffic circle, thereby eliminating the bothersome left turn and enabling the vehicles to move steadily to and from the structure.

In cities, however, the direct street-extension approach has definite shortcomings. A bridge or tunnel is designed to handle continuously moving traffic and can accommodate approximately 1,500 vehicles per hr on each lane, whereas a signal-controlled street has intermittent flow with a lane capacity of from 500 to 800 vehicles per hr. Consequently, an approach that connects directly to a signal-controlled street materially reduces the capacity of the structure. Even with two or three streets at the end of an approach, the cross movements and turning movements of bridge or tunnel traffic, together with local traffic, hinder the access to the structure and limit its lane capacity to from 750 to 1,000 vehicles per hr.

Traffic circles have been utilized to overcome this disadvantage in undeveloped sections of cities, for they enable vehicles to approach the structure from two or more arteries continuously. The Goethals Bridge approach in Staten Island, New York, and the Rockaway Beach approach to the Marine

Parkway Bridge on Long Island, New York, have rotary connections between the approach proper and the connecting roads. This application of the traffic circle is limited to outlying districts, however, because the large part of the unused area in the central island makes rotaries uneconomical in the highly developed part of a city.

The direct street-extension approach develops additional complications on a toll structure, if the collection booths are not placed a sufficient distance beyond the intersection of the approach and the signal-controlled street. The vehicles enter the approach in surges, according to the traffic-light cycle, and the collectors are not able to pass all the cars that move toward the booths on each interval. Consequently, traffic accumulates back across the intersections and blocks the flow of vehicles on the intersecting street.

On the approach itself, off-bound vehicles are required to wait in long queues which sometimes extend back on the structure proper. This condition in undesirable on a bridge, and dangerous in a tunnel. The hazard which may result is illustrated by an experience in the Liberty Tunnel in Pittsburgh, Pa. One evening a tie-up beyond the approach caused a line of stalled vehicles to form back through one of the tubes. Before the policemen could order all the motors to be turned off, the tunnel was filled with carbon monoxide and, although no fatalities resulted, several people were overcome by the gas.

Reservoir Approaches.—To obviate the disadvantages of the street-extension approach, the engineers of the Holland Tunnel in New York, N. Y., developed a reservoir for tunnel traffic (Fig. 1) by clearing the block in front of the portal. Instead of forcing traffic to "worm" its way through a street that crosses the tunnel roadway at grade, they paved the entire block, thus enabling the vehicles to enter the tunnel from four street intersections. By the use of painted lines and channelizing islands they established the bounds of the plaza and directed the movements of the vehicles entering it.

This type of plaza provides a storage space or reservoir wherein 150 to 200 vehicles from the bordering streets can collect without obstructing local traffic. The capacity of the facility is not limited by the traffic-light cycle on any one of these streets, because the alternating green signal permits north and south traffic to enter while those from the east and west wait, and vice versa. On a reservoir plaza, the toll booths are arranged in a circular arc to accommodate the vehicles which enter the plaza at all angles. Consequently, the bridge or tunnel is not wholly subjected to the intermittence of local traffic movement and it is able to accommodate more than 1,200 vehicles per lane in peak hours.

The reservoir approach has been principally applied to tunnels because it permits cars to clear through the tubes and wait on the plazas in the event of an emergency, such as a traffic tie-up. This type of approach is used at the Mersey Tunnel in England, both at the Kingsway entrance in Liverpool and the King's Square entrance in Birkenhead. The over-all shape of these plazas is a sector, with the portal at the apex and the reservoir at the widest part. The entrance and exit facilities are combined on each plaza, and the toll booths are arranged on one quadrant of the semicircular opening. The Antwerp Tunnel under the Schilde River also has a reservoir plaza on one side.

The primary disadvantage of the reservoir approach is that at peak periods it causes all vehicles to stop and wait, or to move slowly and cautiously, before entering the tunnel. Practically all of these approaches are of such shape as to preclude the continuous movement of vehicles into the tunnel after they

FIG. 1.—THE RESERVOIR PLAZA AT THE NEW YORK CITY ENTRANCE OF THE HOLLAND TUNNEL, SHOWING THE SEMICIRCULAR ARRANGEMENT OF TOLL BOOTHS

leave the toll booths. The lanes leading to the portal all converge on one junction together and the angles between these lanes are too great to permit smooth convergence. On the New York entrance plaza of the Holland Tunnel, the twelve lanes are distributed through an angle of 180°. Traffic on the outside lanes makes a right-angle turn to enter the tunnel, a movement which slows down its convergence with the other vehicles. In addition, the twelve radial lines of cars must narrow into two lanes in a relatively short space. During periods of heavy traffic flow, an officer is needed on this type of plaza to direct the entrance movements.

In peak hours, this unwieldy maneuvering often causes traffic to overflow the plazas and collect on the bordering streets. As many as 450 vehicles have been observed waiting to enter the Holland Tunnel, 250 of them off the plaza and obstructing local traffic. Although this condition is likely to occur on any facility in which the volume exceeds the capacity, in this instance it is the inertia of the convergence movements which impedes the flow of traffic and limits the capacity of the tunnel.

Another disadvantage of the reservoir plaza is its ill-adapted use of storage space, particularly on the square or rectangular plazas. Even when crowded, only about 60% of the reservoir area is usable for storing cars. In the business or industrial section of a city, it is uneconomical to waste such a large percentage of available area.

Tapered Plazas with Feeder Connections.—Recognizing that the railroads had been coping with the problem of converging and diverging operations at terminals for almost a century, J. C. Evans, M. Am. Soc. C. E., applied their solution to the approaches of the George Washington Bridge.

The plazas became combination advance and receiving yards, the adaptation of which Mr. Evans describes as follows:[3]

> "* * * the advance yard is made up of definite tracks, accommodating trains and units of equipment from each converging line or common point of origin, without fouling the clearance points of following traffic.
>
> "Such a layout provides a gradually tapering area from which traffic flows without abrupt changes in direction in regular routine, and into which traffic enters upon its proper divergent path.
>
> "It has been demonstrated that capacity performance of river crossings in peak with vehicles properly spaced and travelling at authorized speeds are dependent upon entrance facilities and the proper assimilation of vehicles from the various lanes of approach. If the changes in direction of travel are too abrupt, interferences occur which cause delays and inefficient performances.
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> "Since maximum flow-capacity depends upon entrance conditions, the approach should be so located with reference to various accesses that no abrupt changes in direction can occur upon it, but should permit the progressive convergence of vehicles from all lanes at acute angles."

FIG. 2.—THE DIAMOND-SHAPED TOLL PLAZA OF THE SAN FRANCISCO-OAKLAND BAY BRIDGE

The efficacy of this analysis led to an almost universal adoption of the tapered plaza. Following its institution on the George Washington Bridge,

[3] "Approaches to River Crossings," by J. C. Evans, an address at Harvard University, May 13, 1938 (not published).

it was constructed on the Golden Gate Bridge and the San Francisco-Oakland Bay Bridge (Fig. 2) in California, and in the Lincoln Tunnel in New York City.

To augment the utility of the tapered plaza, Mr. Evans extended a series of direct feeder connections to the main traffic arteries whereby the bridge traffic was collected and diffused in its line of travel without interefering with local traffic. In cities this method of infiltration to the street system is particularly advantageous because it distributes the volume of traffic over a wide area and accommodates the through movements in the various directions of travel. On the New Jersey approach of the Lincoln Tunnel (Fig. 3) an express highway has been extended westward for 1.5 miles in New Jersey through Weehawken, Union City, and North Bergen with separate ramps for inbound and outbound traffic in each of these communities. Similarly, the San Francisco-Oakland Bay Bridge accommodates its east bay traffic by extending a direct connection north for two miles and another east for one mile from the bridge. These connections not only facilitate traffic in these directions, but liberate the streets in the vicinity of the approach from undue congestion.

This type of approach, the tapered plaza with feeder connections, satisfies the previously cited definition by Mr. Watson[2] most completely because it affords the maximum convenience of access and makes the entire capacity of the structure available to traffic. It is also the most expensive type, since it involves a far greater amount of construction and land acquisition than is required for either a direct extension or reservoir approach. The expenditure warranted to obtain maximum convenience and availability of capacity on the approaches of a bridge or tunnel depends upon the volume of traffic that is anticipated to use the facility during its service life. In this paper, however, the scope of the subject and the limitations of space preclude any detailed discussion of the economics of approach planning, except as it is linked with the discussion of approach connections and plazas which follows.

Approach Connections

All the major streets and highways in the vicinity of a proposed river crossing are potential feeders of traffic to it. One of the primary aims in locating the approaches of a structure is to secure the position that is best suited to the collection and diffusion of vehicles by way of these arteries; and this includes not only existing street or highway systems but any proposed improvements, since the latter may become the heaviest feeders of the structure.

In developing an approach, the objective is to provide a system of well-selected, short trunk connections which will tie these feeder streets and highways to the bridge or tunnel. Each connection should have a complement of branching laterals, placed strategically to the zones of origin-destination. Two criteria determine the development of a system of approved connections: (*a*) The adequacy of the street or highway system; and (*b*) the expenditure warranted in providing time and distance savings to the motorist.

The first criterion indicates the capability of the existing system to feed and diffuse the traffic of the river crossing, whereas the second determines the degree to which improvement of this system is justified. If a toll bridge or

Fig. 3.—The New Jersey Approach of the Lincoln Tunnel Showing the Express Highway Connection Which Passes Over and Under the Local Streets but Connects with All the Main Routes

tunnel is to be utilized fully, its connections should be able to accommodate all of its traffic adequately and, in addition, provide benefits to the motoring public commensurate with the toll.

In developing the fantail of connecting links between the structure and the zones of origin-destination it is necessary, therefore, to:

(1) Determine the geographical distribution of the traffic over the existing streets and highways contributing to the proposed crossing, and the adequacy of these routes to accommodate this additional traffic;

(2) Develop the connections that are necessary for the proper dispatching of the traffic at the crossing at the time it is opened to service; and

(3) Make provisions for the expansion of these approach connections to handle a future increase in traffic and to embrace any proposed highway developments in the vicinity of the structure.

Geographical Distribution of Traffic.—From the origin-destination studies of the traffic on existing river crossings and the determination of the volume that can be diverted to the proposed crossing, a flow diagram can be prepared which will indicate the percentage of traffic terminating in each zone. With this diagram as a basis, and assuming that traffic follows the most direct routes between the structure and the zones, it is possible to select the existing streets and highways over which the various portions of bridge or tunnel traffic will be diffused. This is essentially the same procedure that is used in estimating the time and distance savings in the determination of diverted traffic. If two or more routes are available in any line of travel, a distribution is made among them in anticipation of the motorist's choice. This distribution pattern suggests the routes that are to become the main arteries of traffic to and from the proposed crossing and focuses attention on the connections that should be developed or improved.

A survey is then made on these feeder routes to determine the gross hourly capacity of each and the volume of traffic that uses it normally. In planning the connections of the Lincoln and Queens Midtown tunnels in New York, estimates[4] were made of the lane capacities of all Manhattan streets in the vicinity of each tunnel plaza, appropriate reductions being made for the 90-sec traffic-light cycle, parking at curb lanes, and elevated transit line columns. The one-way crosstown streets were found to have a working capacity of 550 vehicles per hr, whereas the capacity of the avenues ranged between 950 and 1,530 vehicles per hr in each direction. In addition, a volume count was made on each of these streets for the hour of the average week-day during which the tunnel traffic was predicted to be heaviest. The difference between the working capacity and the volume during that hour represented the remaining capacity available to handle tunnel traffic.

The remaining capacity of each of the approach streets and highways is a primary factor in the planning of connections because it is a measure of their capability to absorb the traffic over the crossing. By combining these data with the proportional share of bridge or tunnel traffic that is estimated to use

[4] "Report of Probable Traffic Requirements of Queens Midtown Tunnel and Its Arterial and Approach Street and Highway Requirements," by Nathan Cherniack, Assoc. M. Am. Soc. C. E., September 30, 1937.

each route, the points of high concentration and possible congestion may be determined. These locations can then be improved, or connections to alternate routes developed, so that the accessibility of the crossing will be maintained at all times. In numerous cases, the arteries contiguous to the plaza have been given too little attention, with the result that traffic has backed on to the plaza during peak volume periods. For this reason, it is necessary to consider every point of high traffic concentration in the existing system which is to be used for approach and to make the proper deductions for every obstruction to free flow on these feeders. The potential traffic of a vehicular crossing can only be developed when there is sufficient capacity on its approach roads to distribute it adequately.

Development of Connections.—Knowing the peak-hour volume in each general direction from the flow diagram and the capacity of arteries available to this traffic, the next step is to develop a system of laterals branching from the plaza which will facilitate the movement of this traffic. Essentially, it is an hydraulics problem of fitting wye, tee, and elbow connections between a high-pressure main and a system of service lines.

In rural or suburban areas, the consideration of sufficient available capacity of existing routes to absorb the traffic from the crossing is not so much of a problem as that of direct, expeditious connection. Greater benefits to the potential bridge or tunnel patronage in time savings, distance savings, and convenience, can be afforded here and it is desirable to focus attention on this phase of the approach development. A larger and more extensive network of connections can be provided which will draw traffic from all important arterial routes. The New Jersey approach of the Lincoln Tunnel (Fig. 3) is an example of a fast, convenient system of connections which converges all the nearby roads to the tunnel.

In large cities, however, the cost of real estate and the interference with other structures generally prohibits the provision of any but the minimum number of separate connections required to "nose" each vehicle in its proper direction. The objective is to distribute the fixed crossing traffic on the streets in the vicinity of the plazas in accordance with the geographical distribution as previously determined and with due regard to the available margins of street capacity in each direction.

In the studies of the Manhattan approach of the Queens Midtown Tunnel, the peak-hour volume was apportioned to four quadrants—northwest, northeast, etc.—from the flow diagram and then distributed among the streets intersecting the plaza avenues. The surplus margin of street capacity was determined for each quadrant, and although the minimum requirements were not definitely established, the designers were afforded a measure of the adequacy of these connections to serve the tunnel traffic.

The Manhattan approaches of the Lincoln and Queens Midtown tunnels are particularly well adapted to the gridiron street system, because they both provide approach avenues that intersect the crosstown streets (Fig. 4). In developing these approaches, the designers were limited by high property costs to a distributer avenue, from each plaza, of sufficient length to accommodate

the collection and diffusion of vehicles without congestion in the area contiguous to the plazas. Each avenue is precisely long enough to intersect two or three cross streets in each direction and thereby to collect and diffuse the tunnel traffic without exceeding the margin of capacity on any of them. This system is flexible because the motorist has a choice of two or three streets in each direction. If at any time their total margin of capacity should become insufficient to handle the tunnel traffic, the distribution avenues can be extended to intersect additional cross streets.

On the Oakland approach of the San Francisco-Oakland Bay Bridge, the spread of traffic into its lines of travel is accomplished by a branching pattern of short limited-way structures which convey each part of traffic directly to an arterial route (Fig. 5). This differs from the diffusion system of the Lincoln

FIG. 4.—THE NEW YORK CITY APPROACH OF THE LINCOLN TUNNEL, SHOWING THE TAPERED PLAZAS AND THE TWO APPROACH AVENUES WHICH COLLECT AND DIFFUSE TRAFFIC TO THE CITY STREETS

and Queens Midtown tunnels in that it does not distribute each line of travel into the street system multifariously, but conducts it to a main artery in its direction. When it can be afforded, this system of separate connections for each line of travel is more advantageous because it eliminates any possibility of congestion due to cross movements and overlapping of traffic near the plaza. In addition, it offers the advantage to motorists of greater saving in time due to its freedom from the street-signal system.

Numerous combinations of existing street and express connections are possible, the development of each being a matter of economic study. If the volume in any single direction warrants it, a direct freeway connection to the main route in that direction should be constructed. Wherever existing streets

are to be used for connections, however, an adequate margin of capacity must be made available to bridge or tunnel traffic either by widening these streets or by embracing more of them into the approach system.

Provisions for Future Connections.—The limitations in traffic forecasting make it impossible to plan the approach connections beyond the first few years of operation. Nor is it possible to predict which of the extant proposals for street or highway traffic improvement in the vicinity of the structure will materialize. These uncertainties make it necessary to provide latitude for future expansion of the approach connections.

Although the proposed routes of a street or highway development program are important considerations in locating an approach, they become paramount factors in the planning of its connections. The location of a bridge or tunnel

FIG. 5.—THE DISTRIBUTION STRUCTURE AT THE EAST BAY END OF THE SAN FRANCISCO-OAKLAND BAY BRIDGE WHICH COLLECTS AND DIFFUSES TRAFFIC TO THE CITIES OF BERKELEY, ALAMEDA, AND OAKLAND, CALIF.

will often influence the conduct of a local development program. Therefore, some provisions must be planned to incorporate these arteries into the approach system if its utility as a transportation facility is to be maintained. In all of the planning studies for the New York approach of the Lincoln Tunnel, provisions for direct connections to the proposed 38th Street Crosstown Tunnel were included, because traffic studies indicated that 25% of its patronage would use the latter facility if it were constructed.

The increase in traffic on both the river crossing and local streets is another factor which cannot be predicted, but must be anticipated. In determining the

adequacy of local streets to handle bridge or tunnel traffic only the present margins of capacity are calculated. It is essential, therefore, that some real means of relief be planned in the event that these streets become congested by bridge or tunnel traffic in the future. Recognition of the possibility of such a condition led to the inclusion of a provision in the agreement between the City of New York and The Port of New York Authority for the construction of an approach tunnel under 178th Street when the annual volume of George Washington Bridge traffic reached 10,000,000 vehicles. It was estimated that the local streets between the bridge plaza and the eastern side of Manhattan would not accommodate the proportional share of traffic in that direction in excess of that amount. By deferring the opening of this connection until it is needed, considerable savings are made and the surplus revenues from operation of the bridge are available to finance the improvement.

Types of Connections.—In this paper it is not possible to discuss all the possible street and highway improvements, relocations, and new developments which may be utilized in a system of approach connections. For rural areas, the standards of the American Association of State Highway Officials indicate modern practice and they are the best guide. No similar standards are available for urban connections but there are numerous possibilities, ranging from the widening of local streets to the construction of express highways.

Two general methods of handling traffic in cities have proved sufficiently effective to warrant consideration of them for connections to bridges and tunnels. These are the steady flow system, developed by Fritz Malcher, and the freeway as expounded by Miller McClintock.

The steady flow system[5] consists primarily in the elimination of signal control, left turns, and right-angle intersections by an application of the rotary principle. A series of elongated islands, 10 to 40 ft in width and 200 to 500 ft in length, form a medial strip along the center of the roadway and separate the vehicles moving in opposite directions. The islands extend entirely across each intersection, thereby preventing any vehicles from crossing the roadway at right angles, or from making a left turn. A motorist who wishes to cross the road, or to go left on it, must turn right into the main traffic stream and weave across it diagonally until he reaches the turning roadway between the islands. After passing through the medial strip, he enters the stream in the opposite direction by a right turn and can then continue in this direction or turn off on the intersecting street. All vehicles crossing a steady flow connection must make this U-turn on it for half a block or more.

The steady flow system is applicable to streets that are being widened or improved to accommodate a large volume of bridge or tunnel traffic, in addition to local traffic, and it is particularly useful when there is not much cross traffic. Its exponents claim that it increases the capacity of each lane on the roadway 132% over that under the signal-control system; and there is little doubt that traffic can move faster when it is freed from the delays caused by stop-and-go signals. This system of traffic control is used effectively in connecting the Marine Parkway Bridge to Beach Channel Drive in Rockaway Beach, Brook-

[5] "The Steady Flow System," by Fritz Malcher, Harvard Univ. Press, 1935.

lyn, N. Y. The East River Drive leading to the Triborough Bridge in New York City is a modification of the steady flow system, although it is controlled by traffic signals, and left turns are permitted at a few places.

The second type of approach connection, the freeway or express highway, implies new constructions, for it is generally depressed below or elevated above the existing street system. As described by McClintock,[6] it embraces the following physical elements:

> 1. "Complete and continuous physical separation of opposed streams of traffic by a medial strip.
> 2. "Elimination of direct access from abutting property to the roadway and restriction of entries and exits to a limited number of locations.
> 3. "Complete separation of grade at all intersections with no cross movements across the operating lanes of the freeway.
> 4. "A cross section design permitting adequate segregation of fast and slow vehicles, with accelerating lanes at the entries and decelerating lanes at the exits."

This type of approach connection is the ultimate in safe and expeditious travel because it delivers the traffic in a closed conduit past all conflicts and delays. In this respect, it is similar to the bridge or tunnel itself. Two outstanding examples of this type of approach connection are the extension of Grand Central Parkway to the Triborough Bridge (Fig. 6) and the express highway connecting U. S. Route 1 and Route 3 to the Lincoln Tunnel (Fig. 3). Both of these are flanked by streets on each side to accommodate local traffic and give access to abutting property.

Design of Approach Connections.—The modern car has been developed so completely that it is essential to provide for the utilization of its speed potentialities in the design of a new facility. In many cases, the justification of a new bridge or tunnel lies solely in its ability to offer a saving in time to motorists now using a ferry or congested fixed crossing. Whether this economic advantage is necessary or not, the pressure of modern traffic makes it necessary to consider the concept of time in the geometric development of the approaches to a facility.

Design Speed.—Because every feature of alinement is dependent upon it, design speed is the most significant factor in traffic design requirements. Design speed may be considered as that which will accommodate rapid, expeditious travel and provide an adequate factor of safety. It is the maximum uniform speed at which most drivers will operate on a highway. The Oregon State Highway Commission designs for what it terms a critical speed which is approximately 50% greater than safe operating speed; that is, the various details of alinement, curvature, superelevation, lane width, etc., of a 40-miles-per-hr highway are designed to provide safety for speeds up to 60 miles per hr. Whether this conception is practical has been questioned by several state highway officials. With the large mileage of straight alinement and the limited enforcement, highway speeds are usually determined by factors outside the province of road design. On bridges and tunnels, however, the mileage is

[6] "San Francisco Citywide Traffic Survey," by Miller McClintock, 1937.

FIG. 6.—THE EXPRESS HIGHWAY CONNECTION FROM THE TRIBOROUGH BRIDGE TO THE GRAND CENTRAL PARKWAY IN QUEENS, NEW YORK

relatively short and well policed, so that operating speed is generally a matter of regulation. With this control it is possible to establish a maximum operating speed in advance, add a factor for safety, and thereby obtain the design speed.

It is difficult, however, to assume a design speed arbitrarily until studies have been made to ascertain the maximum operating speed for which it is economical to design the approaches. This will depend largely upon the terrain, the type of connections, and the nature of the surroundings. The connections to a city street system may be designed for only 30 miles per hr, whereas those of a state highway system are designed for 60 miles per hr. In general, the design speed should be great enough so that the operating speed of each connection will not be lower than that of the street or highway which it serves. Lower operating speed for the connection would cause it to function as a bottle neck, thereby producing congestion and delay.

From the standpoint of safety and efficient operation, it is more significant that the design be balanced so that every critical item of the roadway is calculated for the same design speed. Once this speed value has been determined, the curve radii, superelevation, sight distances, pavement widths, wall setbacks, and widening at curves should be designed for it. Any unexpected detail requiring or encouraging a sudden change of speed is not only a disconcertion to the motorist, but an accident hazard as well.

Sight Distance.—The ability to see ahead sufficiently is necessary if the motorist is to assist in maintaining the objectives of safety and efficiency. On a bridge or tunnel the length of unobstructed vision need not be as long as that on a highway designed for the same speed, due to the absence of overtaking and passing. This non-passing sight distance should be sufficient, however, to bring a car to a complete stop when the driver is confronted with a stalled vehicle, pedestrian, or similar obstruction in the same lane.

The uninterrupted linear distance required to permit safe stopping can be determined from the formula:

$$s = \frac{v}{30\,f} + 1.47\,t\,v \qquad \text{(1)}$$

in which s is the sight distance in feet; v is the design speed in miles per hr; f is the coefficient of friction of the roadway; and t is the total perception and reaction time of the average driver, in seconds. Assuming a braking coefficient of friction of 0.4 (which is equivalent to a deceleration of 13.2 ft per sec^2) and a combined perception-reaction time of 3 sec, the sight distance on a roadway designed for 60 miles per hr should not be less than 565 ft. If possible, it is desirable to maintain the sight distance at a minimum of ten times the design speed in miles per hr.

On curves less than 500 ft in radius, it is necessary to recede any side walls, slopes, trees, or shrubbery to provide this sight distance. The setback distance from the curb in each instance will depend upon the sharpness of the curve.

Curvature.—After the general direction of each connection has been established, its location is determined chiefly from right-of-way limitations, controlling grades, and curvature. Knowing the design speed, the limiting curvature

may be obtained from the formula:

$$r = \frac{0.067\, v^2}{E + f_l} \qquad (2)$$

where r is the radius in feet; E is the rate of superelevation; and f_l is the coefficient of friction laterally.

Prevailing conditions in each instance will determine the applicable values of E and f_l. The superelevation can be limited to that which will counteract about three-fourths of the centrifugal force at the assumed design speed, and for practical reasons should not exceed 0.10 ft per ft. Tests to determine the friction that may be used with safety and without feeling side pitch outward have resulted in an acceptable friction factor of 0.15. These values substituted in Equation (2) result in a 430-ft minimum radius for a design speed of 40 miles per hr and a 965-ft minimum for a speed of 60 miles per hr.

On curves of 2,000 ft radius or less, transition spirals are necessary to soften the sudden application of centrifugal force on the vehicle. Otherwise the motorist will make the transition by cutting across the adjoining lane, thereby creating an accident hazard. The length of spiral long enough to ease the transition and gradually develop the superelevation may be computed from the formula:

$$L = \frac{3.16\, v^3}{C\, r} \qquad (3)$$

in which L is the length of spiral in feet, and C is the rate of change of acceleration. Studies of riding comfort on street cars indicate that 2 is the most acceptable value of C. For a design speed of 40 miles per hr, the minimum length of spiral at the sharpest permissible curve is 235 ft; for 60 miles per hr, it is 350 ft.

Grades.—Grades are a controlling feature in the planning and design of a bridge or tunnel because they determine the distance which the structure must be extended inland to reach the surface. Likewise, on the plazas and approach connections, the grades determine the conformity of these parts of the facility to the topography and existing traffic arteries.

The prevailing low power ratio of trucks restrains smooth operation on steep grades, particularly when the facility is carrying a large volume of traffic. The following relationship has been developed by the Oregon State Highway Commission to express the operating speeds of heavy vehicles on ascending grades:

$$v_0 = 60 - 0.5\, W - 4.33\, G \qquad (4)$$

in which v_0 is the operating speed in miles per hour, W is gross weight in thousands of pounds, and G is the percentage of grade. On the Pennsylvania Turnpike across the Alleghany Mountains, the maximum grade has been set at 3% in order that trucks may use this facility to its full capacity. Particular attention must be given to this factor, therefore, if the bridge or tunnel is expected to accommodate an appreciable volume of heavy vehicles.

With mixed traffic, it is desirable to limit the maximum upgrade on the plazas and main connections to 3.5%. If the plazas or connecting roads are separated for traffic in each direction, the downgrade maximum may be established at 4%. Depending upon the importance and length of other connections, these limits may be raised to 5% upgrade and 7% downgrade.

Lane Width.—Considerable study has been made on the width of highway lanes, and although no definite conclusions have been formulated, the indications point toward 10-ft lanes for light traffic roads and 11-ft or 12-ft lanes for heavy mixed traffic. These results are premised upon frequent passing of vehicles and speeds to more than 50 miles per hr, however—conditions which are not normally present on bridge or tunnel connections. Under the operating restrictions on these facilities, 10-ft lanes are normally adequate for safe clearance between all types of vehicles.

Due to the tendency of motorists to shy away from curbs, it is desirable to increase this width to 12 ft on the outside lanes of the connections when shoulders are not provided. The average edge distance, measured from wheel to curb, is 3.5 ft at 40 miles per hr, and it should be compensated in additional width. Similarly, on curves of less than 750-ft radius, the lanes should be widened to provide for the tracking of the vehicles.

Wye Junctions at Connections.—The critical points on every approach are the junctions between converging connections and between the connections and the plaza, for it is at these locations that fouling of converging or diverging streams of traffic is most likely to occur. Any point at which the roadways branch is a potential accident and congestion location, and it is worth while, therefore, to give it particular attention. Since the average motorist cannot be relied upon always to make the snap decision accurately, the choice between connections should be minimized at every junction and should afford sufficient time to the motorist to enable him to direct the vehicle smoothly.

For this reason, it is not desirable to have more than one roadway branching from another at any location. This should be especially guarded against at the plaza, where there is often apparent a false economy in branching off a group of connections together or at close spacing to each other. Adapting railroad location principles to the design of approaches, each connection should branch from the plaza singly and at sufficient distance from the others to be singularly distinguished by the motorist. Likewise, the network of connections should form a pattern of wyes, similar to the branches of a tree. The distance between junctions should be at least 300 ft (and preferably more) to enable the motorist to make the proper selection. This arrangement distributes the points of conflict and minimizes the hazard due to indecision or wrong decision at each junction.

The angle of convergence between roadways determines the smoothness with which the junction is made. It is desirable to reduce, as much as possible, the slowing down of traffic which results when streams of vehicles divide or join at the intersection of two roadways. Therefore, the connections should be designed so that the main part of the traffic follows a straight alinement while the minor part branches off at one small angle between it and the main road-

way, or a right-angle turn, depending upon the design speed to be maintained and the prevailing physical conditions.

In so far as it is possible, a connection should give the motorist some indication of its ultimate direction before he steers into it. Frequently a roadway heads in one direction, and then turns through a 180° or 270° angle which is obscured at the outset. Where existing streets are used for approach connections, traffic is often required to make several turns, aggregating 360° or more, before finally heading in a particular direction. A certain amount of curvature is necessary to integrate the approach system properly; but it should be made visible or should be marked clearly so that the motorist has an indication of the direction in which each connection is heading. The distribution structure at the East Bay side of the San Francisco-Oakland Bay Bridge (Fig. 5), is an excellent example of a complex connection on which little confusion occurs. Despite its number of intertwined ramps, the motorist is always oriented, since each roadway in branching from a wye indicates its ultimate direction.

Plazas

A plaza is constructed at the ends of a river crossing for either of two purposes, namely, to collect or distribute traffic between the crossing proper and its approach connections, and to accommodate the collection of tolls. By recognizing the requirements of these two operations, the arrangement, dimen-

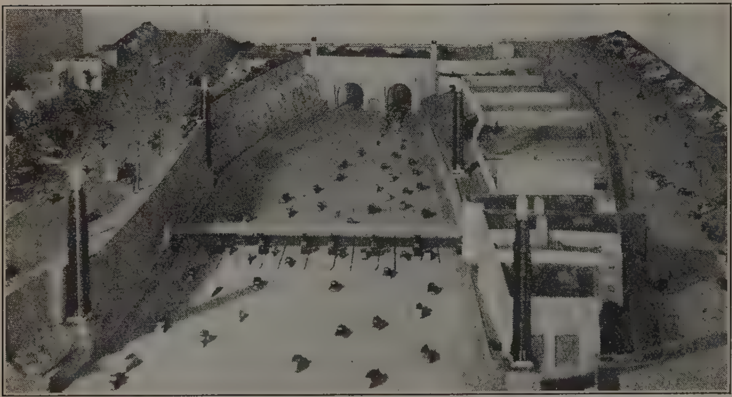

Fig. 7.—A Model of the New Jersey Approach of the Lincoln Tunnel Which Was Used in Studying the Plaza Functions of Toll Collection and Lane Reduction

tions, and other factors relating to the design of plazas may be determined. The extent of development in each application depends upon the volume and concentration of traffic that is expected to use the facility. On the Marin County approach of the Golden Gate Bridge, the plaza was omitted because all tolls are collected on the San Francisco side and there is only one major highway connected to the structure directly. In contradistinction to this, the

New Jersey approach of the Lincoln Tunnel (Fig. 7) has an extensive area, 835 ft in length and 181.5 ft in width at one section, to facilitate the convergence of traffic and the collection of tolls.

Separation of Entrance and Exit Plazas.—Generally the first consideration in plaza design is the arrangement of the entrance and exit facilities. The Holland Tunnel engineers made a forward step in this problem when they decentralized the traffic at both approaches of the tunnel by separating the entrance and exit plazas. This arrangement provides greater access to the tunnel for inbound vehicles, and enables outbound vehicles to disperse more rapidly. Thus far, this method of traffic segregation has been limited to twin-tube tunnels, however, because the separated tubes are more easily adapted to individual plazas at each end than a structure carrying two-way traffic.

The purpose of decentralization is not to keep the entrance and exit plazas apart, but to reduce the overlapping and cross-currents of traffic. Studies of traffic flow at the New York approach of the Lincoln Tunnel demonstrated that unless the two plazas are arranged to minimize this conflict, the congestion in the surrounding area may be as severe as that around a centralized terminal. One approach proposal for the Lincoln Tunnel was rejected when its traffic flow diagram revealed that Tenth Avenue in New York City would have to handle 1,850 vehicles of tunnel traffic at peak hours in addition to its regular volume.

The functions of a particular plaza will generally determine whether it should be designed to handle one-way or two-way traffic. Plazas that serve only to converge traffic between the approach connections and the crossing proper are adaptable to decentralization. However, toll collection is also a primary function of some plazas, and it is desirable from the operating standpoint to have a central arrangement of toll booths. One line of booths for traffic in both directions permits greater flexibility in adjusting the number of booths to suit the demands of traffic in either direction. It is also easier to supervise, and it facilitates the rotation of toll collectors according to traffic requirements. Although the governing conditions in each instance will indicate the most suitable arrangement, it may be stated generally that the plaza on which tolls are collected should be centralized, whereas that which distributes traffic only is adaptable to decentralization. Because of the greater area required, the former is usually located on the side of the river on which property costs are lower. This liberates the plazas on the more highly developed side from toll collection and affords to them greater flexibility in fitting into the street pattern.

Dimensions of Plazas.—By recognizing the requirements of the two functions, lane reduction and toll collection, it is possible to design the plaza for efficient operation economically. The toll collection facilities determine the width of a tapered plaza, and the distance required for the proper convergence of all vehicles on to the bridge or tunnel roadway determines its length.

In like manner, the design requirements of a reservoir plaza may be calculated, although the procedure is generally reversed. Since the available area is fixed by the block to be cleared, it becomes necessary to determine the

arrangement of toll booths which will accommodate both functions to mutual advantage. It has been pointed out that the circular arc layout of booths is not suited to the most efficient movement of vehicles. However, this is not an inadequacy of the booth arrangement, but of this type of approach. If these booths are alined at a sufficient radial distance from the focal point on the plaza, the converging lines of traffic have adequate space in which to merge and the disadvantage of cyclic lane movement is lessened. The semi-circular booth arrangement is best suited to the reservoir type of approach because it is accessible to vehicles entering from three sides, and it spreads them out along radial lines in preparation to being signaled into the tunnel.

The common acceptance of the tapered plaza on the approaches of large river crossings has brought with it the straight-line array of booths across the collection area. The Delaware River Bridge, George Washington Bridge, Golden Gate Bridge, San Francisco-Oakland Bay Bridge, and Lincoln Tunnel each have the toll facilities alined across one of the plazas (Fig. 7). This enables the toll supervisor to control the flow of traffic through each toll lane and to increase the number of booths accommodating a uni-directional peak.

Provisions for Toll Collection Facilities.—The number of lanes through the toll collection area depends upon the mechanics of collection, that is, upon the number of toll classifications and the type of registering equipment. On the George Washington Bridge, Golden Gate Bridge, and similar modern structures, it takes between 6 sec and 8 sec for each vehicle to pass the booth, which is equivalent to from 450 to 600 vehicles per hr. Since the operating capacity of each lane on the crossing is about 1,500 vehicles per hr, three toll lanes are required for each lane on the bridge or tunnel. As shown in Fig. 7, one toll island, approximately 6 ft wide, can accommodate two toll lanes. If these lanes are 10 ft wide and the requisite number of islands is added, it is necessary to provide at least 39 ft at the collection section for each lane on the crossing in developing the structure to its capacity.

Provisions for Convergence of Lanes.—If the plan of the railroad advance yard is to be used in designing a plaza, it is necessary to determine the distance in which vehicles can converge without slowing down traffic. Relatively little investigation has been made on this phase of vehicular operation although it has an important bearing on the requirements for passing, on highways and the design of accelerating lanes, as well as bridge and tunnel plazas. The operations of a car in moving from one lane to another on a plaza are similar to those in passing another vehicle on a highway; and, to a limited degree, this analogy is of value in determining the design requirements for traffic convergence on plazas.

Adequate sight distance is required in both operations because it is no more desirable to converge with limited visibility than it is to overtake another vehicle on a curve. On a bridge or tunnel plaza, the governing conditions are altered, however, because the unforeseen hazard of a vehicle approaching in the opposite direction is not usually present. Th refore, it is unnecessary to provide any more than the minimum non-passing sight distance for the design speed of the plaza.

The converging maneuver of a vehicle is a reverse curve, in which a side movement of one or more lanes is accomplished while the vehicle is traveling forward. The path of the vehicle is actually a transitional spiral throughout, but it may be assumed to be a reverse circular curve with sufficient accuracy, due to the variation in driving characteristics. If the amount of side thrust caused by centrifugal force which the car occupants can comfortably withstand ($f = 0.15$) is not exceeded, the forward distances traveled in moving laterally across a plaza are as shown in Table 1. Observations of passing on highways indicate that, after the two vehicles are abreast of each other, the overtaking vehicle requires an average of approximately 3 sec in which to cross back into lane, with approximately 1 sec extra for each additional lane that is crossed. Although these observations are only available for vehicles moving over three lanes, a total lateral movement of 30 ft, the results may be projected for wider pavements, at a rate of 1 sec extra for additional lane, to obtain a check on Table 1. On this basis, the required lengths for lateral movement at various speeds are as given in Table 2. These two tables check reasonably well, considering the variation in driving habits; and, in the absence of more indicative studies, they may be used to determine the length of plaza required for various widths. For example, the plaza of a four-lane bridge or tunnel, if developed for capacity operation, requires a minimum of 12 toll lanes which with toll booths aggregate a total width of approximately 160 ft. At a design speed of 40 miles per hr, its length from toll booths to portal should be 500 ft

TABLE 1.—FORWARD DISTANCE TRAVELED IN MOVING LATERALLY ACROSS A PLAZA

Speed, in miles per hour	Radius, in feet ($f = 0.15$)	Forward Distance, in Feet, for a Lateral Movement of:							
		10 ft	20 ft	30 ft	40 ft	50 ft	60 ft	70 ft	80 ft
20	179	84	118	143	164	183	198	213	226
30	402	126	178	218	253	279	305	328	350
40	715	169	238	291	336	375	410	442	471
50	1,117	211	298	365	421	470	515	555	593

TABLE 2.—FORWARD DISTANCE TRAVELED IN MOVING LATERALLY AFTER PASSING

Speed, in miles per hour	Forward Distance, in Feet, for a Lateral Movement of:							
	10 ft	20 ft	30 ft	40 ft	50 ft	60 ft	70 ft	80 ft
Equivalent number of lanes..........	1	2	3	4	5	6	7	8
Average time, in seconds..........	3	4	5	6	7	8	9	10
20..............	88	118	147	176	206	235	265	294
30..............	132	176	221	265	309	353	397	441
40..............	176	235	294	353	412	470	529	588
50..............	221	294	368	441	515	588	662	735

in order to permit the vehicles to move laterally from the outer toll booths to the lanes of the structure. This design speed is considerably greater than that at which the vehicles will operate in traveling from the booths to the portal for, at an average acceleration of 2.5 ft per sec^2, they require 690 ft to attain 40 miles per hr. However, the speed of the off-bound vehicles approaching the toll booths is the governing criterion, and it is well to assume the design speed on the plaza to be the same as that at which vehicles are expected to operate on the crossing proper. The New Jersey plaza of the Lincoln Tunnel has thirteen toll lanes and a total width of 181.5 ft. Its length from the booths to the portal is 591 ft. Similarly, the sixteen toll lanes on the San Francisco-Oakland Bay Bridge plaza are converged to six lanes in a distance of approximately 600 ft.

On plazas constructed solely for toll collection, the convergence distance on each side of the booths should be equal since the movements of convergence and divergence are identical on both sides. This type of diamond-shaped plaza was built on the Oakland approach of the San Francisco-Oakland Bay Bridge (see Fig. 2).

Plazas which only assemble or disperse traffic to connections are generally not as long or as wide as those on which tolls are collected. Since traffic does not have to stop, it is only necessary to provide adequate space in which to secure the proper lane reduction between the connections and the roadway of the structure. These plazas are designed according to the principles of convergence distance, as outlined herein, each application being dependent upon the number and type of connections.

Reservoir Space.—The principal advantage of the reservoir type of approach is its provision of adequate reservoir space to take care of the fluctuating surges of traffic to and from the streets during peak periods. As previously stated, this provision is not necessary if the approach is designed to handle traffic without storing it, since each section is capable of handling all the traffic that passes it without interruption or delay. The storage capacity need only be considered in so far as a reservoir may be necessary due to an irregularity in the flow of traffic or as a safeguard in an emergency.

Certain practical limitations to the uninterrupted flow of vehicles may occur, particularly at points of convergence or divergence. The tendency of drivers to slow down while converging from one lane into another may throttle the traffic behind that point at times. Similarly, an emergency, such as a fire or accident, will slow the velocity of traffic on the plazas, and occasionally will cause it to store there temporarily. Lift-span bridges require a reservoir space, normally, and it is also useful in the classification of slow and fast traffic when high-speed and low-speed lanes are provided on the main structure. In spite of these, however, it is seldom necessary to design the plazas for a predetermined storage capacity, since the space provided for convergence and divergence is a more than adequate reservoir.

Portal Transition.—It takes the average motorist about 5 sec to overcome, completely, the temporary blindness that occurs at the sudden transition from a plaza into a tunnel. In average daylight, the level of light intensity on the

plaza is several thousand lumens per square foot, whereas in the tunnel this level is less than 40 lumens per sq ft. The retinal adaptation to this change is a function of the photo-chemical action of the receptor neurons of the eye, which physiologists have found to be dependent not only upon the difference in intensity levels, but, more importantly, upon the suddenness of change in levels.

For this reason, it is essential that the transition between the covered and the open roadway be made as gradual and protracted as possible. The designers of the Lincoln Tunnel have achieved this gradual transition in a measure by flaring the tunnel ceiling upward at the ends of each tube. With the higher opening, more natural light is admitted and the effect is a higher intensity level at these sections, which enables the motorist's eyes to make the retinal adaptation more readily.

The proposed Delaware River Tunnel will also have a flared ceiling at each end, and, in addition, the designers plan to use higher candle-power lights at the portals. This effort at protracting the transition is commendable, but the possibility of producing the reverse effect at night must also be guarded against. On the plaza the illumination may be reduced to only 0.1 lumen per sq ft on a dark night and the sudden change from the well-lighted tunnel will produce the same blindness.

No matter how gradual this transition is, the portal is a critical location. It is necessary, therefore, to make every effort to safeguard the flow of traffic and to minimize the possibility of accidents. In this transition zone which may be considered to extend 100 ft on each side of the portal, any obstructions, changes, or curves are undesirable because the motorist has little opportunity to perceive them sufficiently in advance. The transitions of the Holland and Lincoln tunnels are subject to criticism in this respect because a relatively sharp curve is begun just at the portals in both structures.

Pedestrian Crossings.—Whenever a plaza is to be constructed in an urban district, it is advisable to make a survey of pedestrian movements in the immediate vicinity and to incorporate in the design some means of re-routing this foot traffic if the volume is appreciable. In the highly developed sections which envelop the Holland Tunnel, Delaware River Bridge, and similar approaches, it cannot be expected that the large expanse of plaza area will remain clear of pedestrians unless crossings are provided. Rather than detour 500 to 1,000 ft around the plaza, they prefer to take the chance of "jaywalking" across it. Observations at busy street corners indicate that the percentage of "jaywalking" and irregular pedestrian movements increases with the volume of pedestrian traffic. A small number of people is generally orderly; but as the volume increases, they become increasingly more unwieldy to control. Therefore, the disorder and accident hazard is progressively greater.

Manual control is effective if enough officers are assigned to this duty, but its cost exceeds that of providing a separate pedestrian crossing. It is usually cheaper to install an over- or under-pass for any volume of traffic that will warrant policing. The type of pedestrian crossing depends, of course, on the prevailing conditions at the plaza. One method of subvention is to construct

a walkway over the toll booths. Since the booths are generally roofed, it costs only slightly more to lay a sidewalk pavement on them and to construct the stairways at each side of the plaza.

Another situation which causes an inconvenience to motorists and an accident hazard to pedestrians is the stopping of buses at the plaza to load and unload passengers. When a river crossing is opened initially, bus stops are usually prohibited, but quite frequently the volume and character of bus transportation using the crossing has increased to the point where a stop on the plaza is necessary. Ordinarily, it is not possible to forecast the trend in common carrier service sufficiently to be able to design facilities for these vehicles at the time of initial construction. When the volume of bus transportation does become sufficient to warrant a stop at the plaza, however, a bus station should be constructed nearby. This will permit the buses to clear the plaza and still accommodate their passengers, without disrupting the vehicular operations on the plaza.

Architectural Treatment.—The imposing nature of bridge and tunnel plazas demands that due consideration be given to the amenities of these structures and the contiguous properties. Since the period of the Italian Renaissance, the ample spaces at bridge-heads have been treated by architects for formal beautification. Notable present-day examples are the decorative bridge plazas of Pont Neuf and Pont de la Concords over the Seine in Paris, France.

It is wholly desirable to adorn the plazas of a bridge or tunnel so that they will be attractive. The value of such treatment is not only reflected in the pleasant reaction of the driving public, but also in the enhancement of the surroundings.

In this generation, however, the speed of traffic has increased to the point which precludes the opportunity for automobile occupants to appreciate any extraneous adornment. The stress has been on the practical rather than the beautifying features of plazas and the result has been an adaptation of esthetic proportion consistent with their usage. In developing the amenity of a bridge or tunnel plaza, the functions of the plaza should be kept in mind because the complexities inherent in high-speed operation demand the constant vigilance of the motorist. Any extraneous treatment, such as statues, fountains, etc., are distracting and should be avoided or placed out of the driver's view. Numerous architectural designs have been developed in which a pleasing effect is created as a whole without directing attention to any particular part of it, and it is well to retain this type of treatment on bridge and tunnel plazas.

Conclusion

For a particular bridge or tunnel, the selection of a type of approach, or the development of approach connections and plazas, which is best suited to traffic demands is a matter of economics and engineering judgment. This paper contains some suggestions of the more significant elements, and their places in the design of bridge and tunnel approaches. The important point to be gathered is that the approaches are the valves which control the capacity and the accessibility of the structure. As it has been stated by Commissioner

Robert Moses, executive officer of the Triborough Bridge, Whitestone Bridge, Henry Hudson Bridge, and Marine Parkway Bridge:[7]

> "It is only in recent times that those advocating new bridges and tunnels, and the public officials responsible for voting money and contracting, have been willing to concede that generous approaches and connections to main arteries are even more important than the crossings themselves, and that there is no justification for building these structures without providing adequate means to get to and from them. The building of approaches as an aftermath is always expensive and frequently impossible, but it has been up to now the rule and not the exception."

ACKNOWLEDGMENT

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[7] *New York Times*, July 9, 1939.

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PAPERS

TREND IN HYDRAULIC TURBINE PRACTICE

A SYMPOSIUM

Note.—Written comments are invited for immediate publication; to insure publication, the last iscussion should be submitted by **March 15, 1940.**

ECONOMIC PRINCIPLES IN DESIGN

By I. A. Winter,[1] Assoc. M. Am. Soc. C. E.

Synopsis

New and interesting advancements in hydraulic turbine practice and research, many features of which are still in the proposed or experimental stage, are presented in this paper. The subjects cover a wide range and are treated only in their broadest aspects, based on the experiences of the writer and his associates; it is not intended to present an exhaustive treatment of any subject or to cover the experiences of other qualified experts in the field.

It is hoped that the subjects presented will invite others to contribute constructive data or experiences which will add to the interest of the paper and point toward the future trend in hydraulic turbine practice.

Introduction

Advancements in the application of hydraulics to the design of power plants have not been all that was desired because there has been too little appreciation of the gain in efficiency, and the saving in first cost, made possible by the application of fundamental elementary principles. In general, the designer has attempted to simplify the field construction by approximating shapes that conform to the ideal shapes. He has reasoned that refinement in design was not necessary; and, as a result, the value of the theorist has been minimized as being impractical.

Progress is now being made toward improved designs and reduced cost, stimulated in some degree by the research in aerodynamics which is being conducted by the airplane industry. In that field the speed and load capacities have been increased steadily by lending attention to fine details. In the future, progress in water-power designing will be parallel to the progress in aeronautical design, inasmuch as specific speeds and velocities now in common use will be increased, resulting in higher efficiency, lower maintenance, and reduced first cost.

Turbines

In turbine design the trend is toward larger sizes, of which the Wheeler Dam in Alabama, Bonneville Dam in Oregon, and Hoover Dam in Arizona and Nevada, are outstanding examples. At Wheeler the turbines (with fixed-blade propellers) deliver 45,000 hp at 85.7 rpm, under a head of 48 ft. The turbine runner is 22 ft in diameter and discharges 10,600 cu ft per sec at full gate. The Bonneville turbines are fully automatic Kaplan wheels rated at 66,000 hp under 50-ft head, with a five-bladed runner 23 ft 4 in. in diameter, operating at 75 rpm. At the rated heads and power each wheel discharges

[1] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

13,000 cu ft per sec. Complete tests have been made on the wheels, and the results demonstrate that the intake, scroll case, turbine, and draft tube are well designed. Efficiency appears to be (and capacity is) above expectation, with freedom from hydraulic disturbances. The turbines at Hoover Dam are rated at 115,000 hp, under a head of 475 ft, and they operate at 180 rpm. The maximum discharge is approximately 2,500 cu ft per sec when delivering full power at full gate. Under test conditions these units have delivered 100,000 kw for a short interval at a head of 520 ft, which is approximately 130,000 hp in a single-stage runner.

Many improved features of turbine design are contributing to better performance of the units and to reduced maintenance cost. Venting the center of the draft tubes of fixed-blade Francis type, and propeller runner turbines, with free air through the hollow bore in the turbine shaft, similar to that at the power plants at Hoover Dam and Drop No. 4 of the All-American Canal system in California, is effective in eliminating hydraulic shock in the draft tube caused by vortex cavitation. The admission of free or compressed air into the draft tube is effective in obtaining smooth operation of the unit, regardless of the tailwater level. The quantity of air required to quiet the draft tube in this manner is relatively small and has been found to have no adverse effect on the efficiency or horsepower of the turbine at any gate opening.

Water-lubricated bearings of rubber, lignum-vitae, or non-metallic composition are meeting with favor for propeller-type turbines on both the fixed-blade and the movable-blade types. Provisions for the adjustment of the runner clearances are customary. By using water-lubricated bearings in lieu of oil-lubricated babbitted-type bearings, the inaccessible stuffing box below the bearing is eliminated, and the turbine pit may be drained entirely by gravity. Corrosion-resisting steel sleeves are fitted to the shaft opposite the bearing and stuffing box.

Improvements in the type of gate linkage arrangement have been of importance. A design that incorporates a cast-iron brake element in tension, regardless of the direction of operation, as well as gate-limit stops, independent of all associated parts, has been furnished for the Drop No. 4 plant and is of a type similar to that used extensively in Europe.

Progress is being made in the elimination of pitting on runners by reducing the overhang of gates over the curved entrance to the throat ring, finishing the entrance and discharge edge of the blades, and introducing improved blade shapes of simple curvature. On several new projects, consideration is being given to welding a $\frac{1}{8}$-in. layer of stainless steel on the runner shroud and low-pressure areas of the vanes of a Francis runner in the manufacturer's shop. Under these conditions the steel coating can be applied at less cost than when applied in the field, and the reduced friction losses, obtained by the use of stainless steel with ground surfaces, will increase the power and efficiency of the unit.

Increased attention is being given to the prevention of vibration, noise, and instability of operation. At least eight distinctly different types of trouble from this source have been encountered and satisfactory solutions found. This is a

broad subject, involving many phases of power-plant designs, and offers a fruitful field for further investigation.

One of the outstanding developments in turbines has been the automatic device by which the turbine blades can be moved without hydrostatic pressure from the governor oil system. The blades are designed so that they are pivoted ahead of centers of pressure and are proportioned so that they have an inherent tendency to adjust themselves to the water flow. The adjusting mechanism is concentrated in the runner and turbine pits and is not connected with the governor pressure system. The blades open when the turbine is started to reduce hydraulic thrust, and run-away speed is reduced if the machine should run away at normal head. This unit is a distinct American development.

The head limit for the propeller type of turbine is being increased. Low-speed propeller runners, with from eight to ten blades, have been developed for high heads. This improvement extends into the field of the Francis turbine for specific speeds from 80 to 100, and thus it bridges the gap between the medium-speed Francis turbine and the high-speed propeller turbine. The characteristics of this type of turbine are better and the efficiency is greater than either the high-speed Francis or the usual propeller type.

Laboratory tests on movable-blade propeller turbines have demonstrated that increased power and efficiency at the larger gate openings may be obtained without the necessity of making the bottom of the throat ring spherical. It has been general practice, in the past, to maintain close clearance throughout the periphery of the runner blades when they are opened. The spherical throat reduces the capacity of the runner and causes undesirable eddy formation at the periphery of the draft tube immediately below the runner. It is also conceivable that vortices set up by the wicket gates will have a tendency to collapse on the bulged part of the spherical throat ring, causing increased pitting.

Centrifugal Pumps as Turbines

Laboratory investigations conducted in 1939 by the California Institute of Technology at Pasadena, Calif., under the direction of the United States Bureau of Reclamation in connection with the pumps for the Grand Coulee pumping plant at Coulee Dam, Washington, emphasize the fact that, when operating in reverse as a turbine, a centrifugal pump may be slightly superior in efficiency to a hydraulic turbine designed as such. Such a finding may be quite consistent with the design principles as the friction surfaces in a centrifugal pump are reduced to a minimum. The blades are smaller in number, the direction and changes of flow are less abrupt, and the leakage area of the seal is reduced by placing the seal rings at a smaller diameter than is possible in the conventional turbine design. The significant fact revealed by these tests is that when the pump is operating in reverse the speed and power generation are comparable to that of the pump, thus making an ideal unit for pumped storage projects. The efficiency of the unit when operating as a turbine or pump under the net effective head is approximately the same.

Many years of research and study have been devoted to designing a turbine that could be used as a pump; but the belief that pumps were inherently low in

efficiency precluded any serious research to develop the pump as a turbine. The tests of the pumps for the Grand Coulee pumping plant indicate a definite increase of pump efficiency with an increase of specific speed to a value of approximately 2,500, which is in substantial agreement with the relation of capacity and specific speed to efficiency as shown[2] in Figs. 1 and 2. Fig. 1 shows

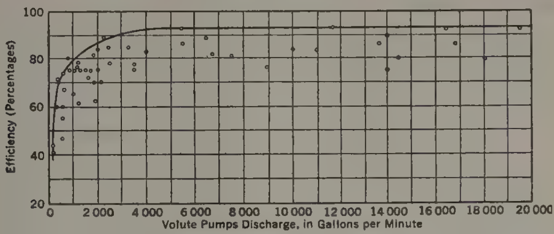


FIG. 1.—OPTIMUM EFFICIENCY AS A FUNCTION OF CAPACITY

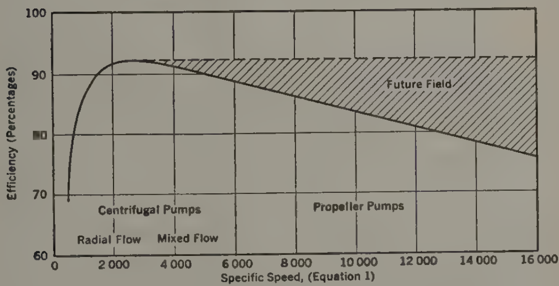


FIG. 2.—OPTIMUM EFFICIENCY AS A FUNCTION OF SPECIFIC SPEED

the optimum efficiency as a function of the capacity of the pumps. This characteristic is not apparent with respect to an increase in specific speed as shown in Fig. 2 where the specific speed N_s is represented by

$$N_s = \frac{N\sqrt{Q}}{h^{3/4}} \qquad (1)$$

in which N = speed; Q = flow, in gallons per minute; and h = hydraulic head. The shaded area marked "future field" is predicated on the assumption that the increase in demand for pumps of higher specific speeds will result in increased performance similar to recent advancement in the characteristics and efficiency of the propeller-type turbine.

In general, the relation of efficiency to specific speed for turbines is approximately parallel to that for centrifugal pumps when expressed in the same unit, maximum efficiency occurring at N_s = 2,500 for single-stage, single-suction pumps; and, N_s = 55 for similar hydraulic turbines. This relation is being modified as will be noted by reference to Fig. 3, showing the performance of the

[2] "Hydraulics," by R. L. Daugherty, McGraw-Hill Book Co., Inc., New York, N. Y., and London, England, 4th Edition, 1937, Figs. 315 and 316.

station-service turbines tested at the Bonneville project from February 5 to 14, 1938. The turbines may be described as follows:

Type: Kaplan, five blades	Scroll: Plate steel
Diameter: 81 in.	Rating: 5,000 hp at 50 ft head
Speed: 257 rpm	Specified Head: 30 ft to 80 ft
Draft Tube: Elbow type	Discharge measured by salt-velocity method

These units have a peak efficiency of 93% when operating at $N_s = 140$. This value compares with the best efficiency obtained on turbines with a specific speed of 55.

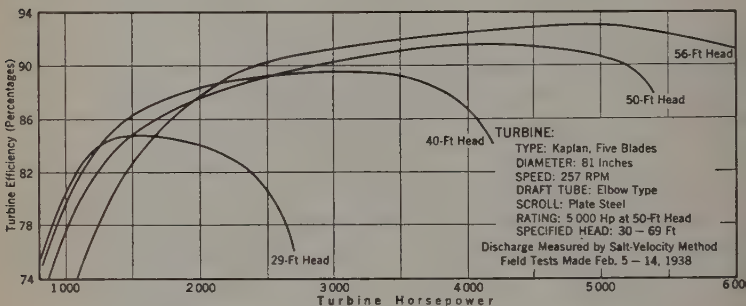


FIG. 3.—FIELD TEST OF STATION-SERVICE TURBINE AT BONNEVILLE

Cavitation, which has been one of the limiting features of specific speed, must be considered as a factor to be solved coincidently with the associated problem. It is conceivable that a steady increase in speed and efficiency, and a reduction in cost, will follow improvements in the shape of the turbine runner vanes and venting of the draft tube, and by the use of hard metals with a high yield point to resist cavitation.

The pumped storage possibilities are exceedingly great for many of the present developed properties where the reservoirs and transmission lines are now in service. The generation of power on the large canal systems which are now being built (1939) throughout Western United States can be made of great economic value by the location of daily pumped storage plants at intervals along the canal system. The All-American Canal system in the southwestern part of the United States will require a seasonal flow variation from a minimum of approximately 3,000 cu ft per sec to a maximum of approximately 6,000 cu ft per sec. A typical power study for this project is shown in Fig. 4. A canal system of this nature does not provide even hourly storage to meet the varying daily load curves; hence, a turbine selected for this class of service must be capable of delivering only base-load power which, in many cases, results in a loss of 20% in the generation of secondary power for which the market has no particular use.

By applying the pumped storage principle to constant varying flows in extended canal-system projects, an ideal power set-up can be created, whereby the economics of the project may be tremendously increased and the hydraulic

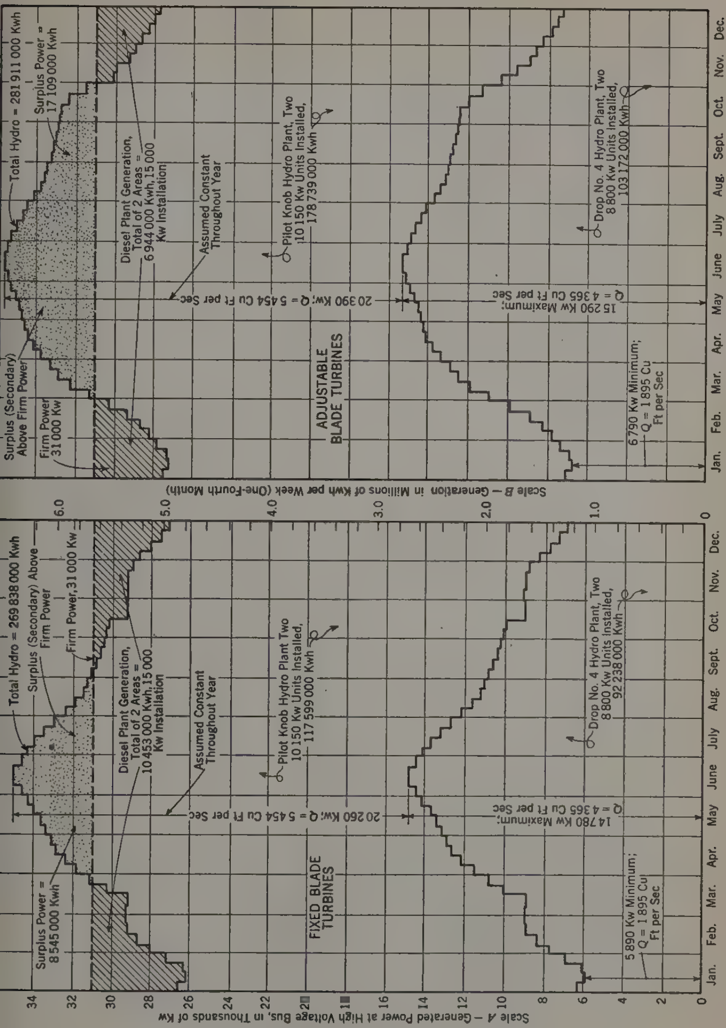


Fig. 4.—System Yearly Powergraph—Using Fixed-Blade and Movable-Blade Turbines

installation in the power plant may require an entirely different basis than would be the case if no storage facilities were provided. A point in question is the selection of a movable-blade, propeller, turbine runner versus a fixed-blade turbine runner to obtain the most economical installation. A study of the absorption of power in a system generated by the fixed-blade turbine may indicate that this type of equipment is economical since the additional power generated by the movable-blade turbine is only secondary power, for which there is no market. An example of this condition is illustrated in Fig. 5, in

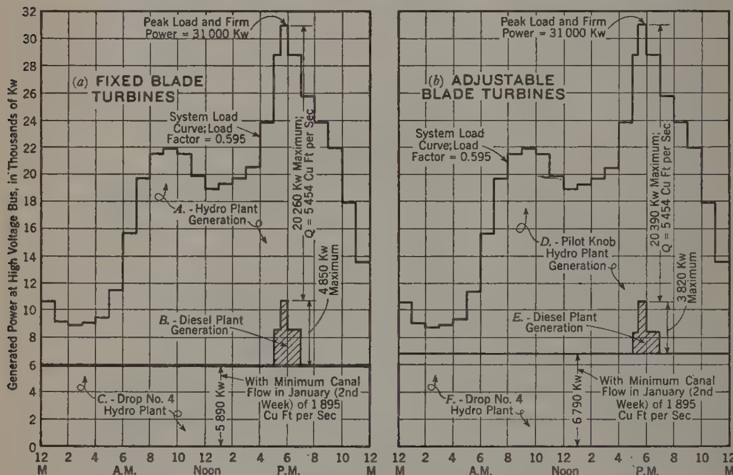


FIG. 5.—SYSTEM DAILY LOAD CURVE—USING FIXED-BLADE AND MOVABLE-BLADE TURBINES

which areas *A* to *F*, inclusive, are further described as follows:

Area (Fig. 5)	Description
A	Hydro plant generation at Pilot Knob Plant, two to ten 150-kw units installed; 295,000 kw-hr.
B	Diesel plant generation; 15,000 kw units installed; hatched area under the curve denotes 6,400 kw-hr.
C	Hydro plant generation, Drop No. 4; two 8,800-kw units installed; 141,360 kw-hr.
D	Hydro plant generation, Pilot Knob; two to ten 150-kw units installed; 275,380 kw-hr.
E	Diesel plant generation; 15,000-kw units installed; hatched area under the curve denotes 15,000 kw-hr.
F	Hydro plant generation, Drop No. 4; two 8,800-kw units installed; 162,960 kw-hr.

The maximum water use for Drop No. 4 and Pilot Knob, at peak load, equals 1,895 + 5,454 = 7,349 cu ft per sec.

The pumped storage plan offers a possible solution of the foregoing apparently inconsistent answer, as full utility of all the power will be possible. This reasoning also applies to the selection of generator capacity to be installed in a plant. The trend will be to increase the total installation materially in order to utilize the available river or canal flow as fully as possible.

Intakes and Penstocks

The intake losses for penstocks, including trashracks and other entrance losses, vary from 0.1 ft to 1.0 ft, with 0.5 ft representing usual losses down to the turbine for medium-head plants with short penstocks. In the designs of intakes for the Wheeler power plant, situated on the Tennessee River, careful consideration was given to the shape of the intake with respect to obtaining flows normal to the line of trashrack and eliminating eddies behind the supporting structure of the trashrack. The struts were streamlined and the section was made to converge so as to obtain the same areas at the end of the struts as the minimum area of the passage and thus pinch off the formation of the eddy current tending to form at the trailing edge of the supporting struts. Under field test this intake showed a gross loss of 0.1 ft at a flow of 9,400 cu ft per sec, corresponding to the point of best efficiency of the turbine. The velocity of water through the racks was 5 ft per sec at full load, contrasted to the usual practice of providing area equivalent to a velocity of 2 ft or 3 ft per sec for the designs that have measured losses that range between 0.5 ft and 1 ft.

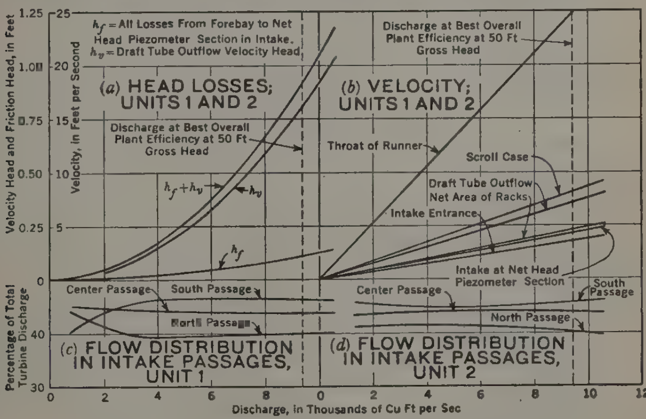


Fig. 6.—Hydraulic Performance Tests; Wheeler Power Plant

The field-measured characteristics for this intake are shown in Fig. 6, and a typical cross section through the plant (except an overhead 270-ton traveling crane) is shown in Fig. 7. In Fig. 6(a), h_f = all losses from the forebay to the

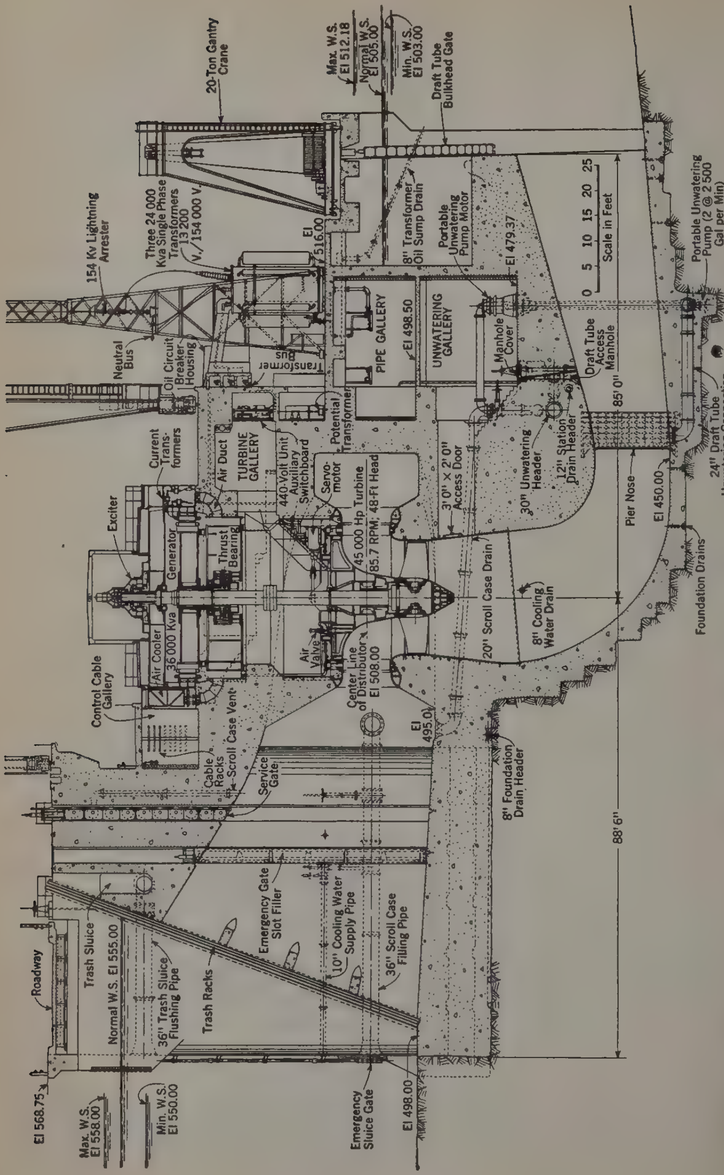


Fig. 7.—Typical Cross Sections through Wheeler Power Plant

net-head piezometer section in the intake; and h_v = the outflow velocity head in the draft tube.

A similar intake was designed for the power plant at Drop No. 4, on the All-American Canal. This intake, complete with homologous turbine, was built and tested in the turbine manufacturer's laboratory and the total loss in head, including 80 ft of penstock, was determined as 0.18 ft, corresponding to the point of best efficiency on the turbine. Stepped up to prototype value, this head loss is approximately 0.15-ft total, including trashrack, supporting piers, entrance losses, and friction losses down to the turbine casing.

This type of large-volume, low-head intake was developed to reduce costs and improve efficiency. The cost was reduced by using smaller rack areas and increasing the apparent velocity through the trashrack structure; and the efficiency was increased by making the full area of the intake and trashrack effective. With the usual bell-mouth intake it is frequently necessary to install a false roof over the upper bell-mouth to obtain satisfactory flow conditions when current meters are being used in the intake section for measuring water. The improved intake, with trashrack symmetrical and normal to the center line of the penstocks, follows the experience gained by the use of these false roofs. Accelerating the flow and streamlining the struts seems to offer a satisfactory solution for the prevention of eddy formation behind supports placed in water passages. This principle of design has been used in various structures and has proved uniformly successful.

A type of intake structure with circular trashracks has been developed for the Grand Coulee power plant on the Columbia River in which all losses from forebay to the circular section of the penstock will not exceed 15% of the velocity head at this point. These intakes, as shown in Fig. 8, incorporate the streamlined feature of the trashrack structure and symmetrical intake normal to the center line of the conduit. The entrance to the penstock is further improved by using the rate of acceleration produced by the area of a *vena contracta* nozzle under spouting velocity. This design is predicated on the assumption that any increase or decrease in area of the approach transition from the gate section to the penstock, other than that of the *vena contracta*, will result in losses due to turbulence as the normal rate of acceleration is destroyed. These conclusions have been borne out by laboratory tests.

Where the center line of the penstock is not normal to the face of the dam, the intakes are further improved by introducing a velocity acceleration component to produce a change in direction of the flow into the penstock. This is accomplished by the construction of a velocity-vector diagram which gives the final velocity as a function of the initial velocity for a given change in direction, as shown in Fig. 9. The increased velocity required to produce the change in direction of flow is superimposed on the acceleration required by the *vena contracta* in accordance with Newton's second law of motion. The areas and velocities required for a transition of this design are shown in Fig. 10 and Table 1. The application of these principles of design results in an intake with the highest possible efficiency and uniform distribution of flow in the

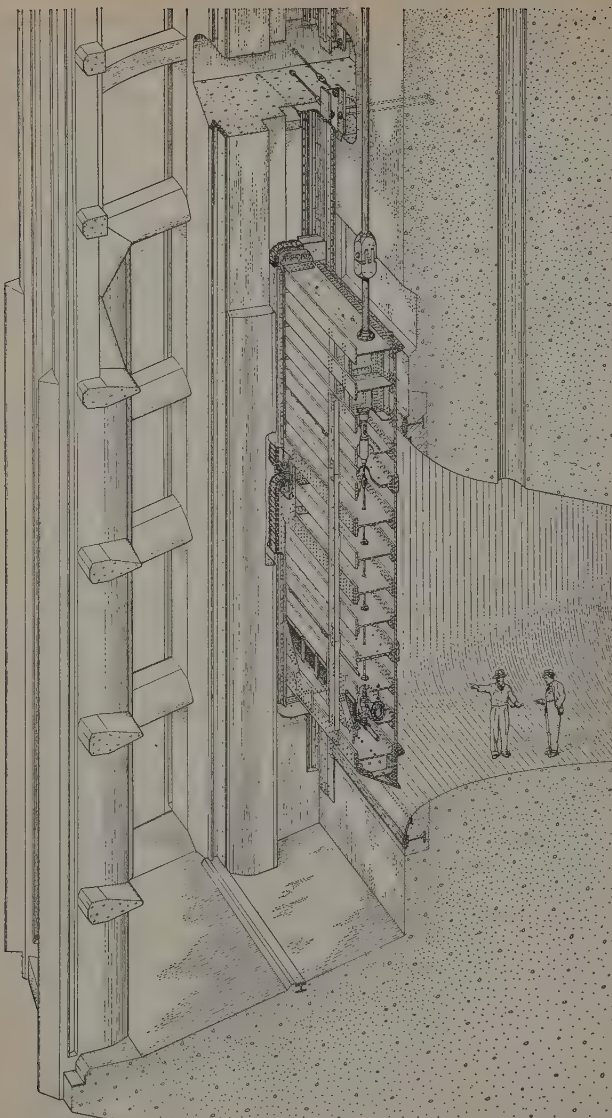
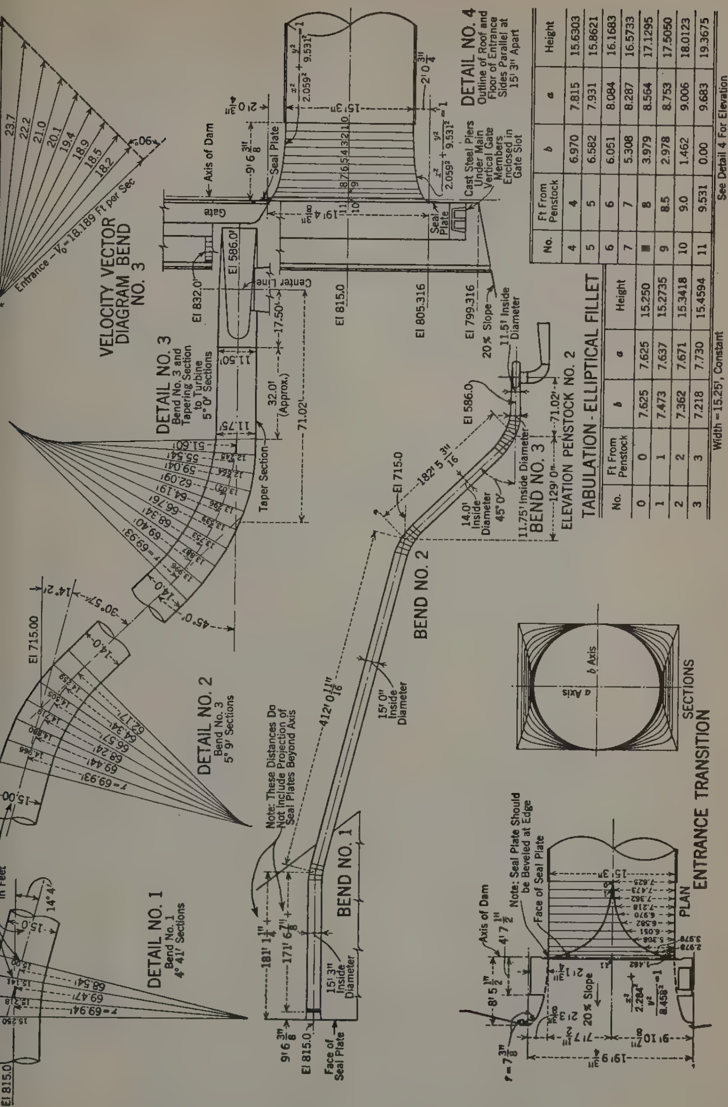


FIG. 8.—PENSTOCK INTAKE, TRASH-RACK, AND GATE FOR THE GRAND COULEE POWER PLANT



TABULATION - ELLIPTICAL FILLET

No.	Ft From Penstock	b	a	Height
0	0	7.625	7.625	15.250
1	1	7.473	7.637	15.2735
2	2	7.362	7.671	15.3418
3	3	7.218	7.730	15.4594
4	4	6.970	7.815	15.6303
5	5	6.582	7.931	15.8621
6	6	6.051	8.084	16.1683
7	7	5.308	8.287	16.5733
[illegible]	8	3.979	8.564	17.1295
9	8.5	2.978	8.753	17.5050
10	9.0	1.462	9.006	18.0123
11	9.531	0.00	9.683	19.3675

Width = 15.25', Constant

See Detail 4 For Elevation

FIG. 9.—HYDRAULIC DESIGNS FOR TAPERING PENSTOCK

conduit and reduces the size of the penstock gates as much as 30% over conventional designs in which entrance velocities of from 5 to 7 ft per sec are selected. Not only do apparent low-velocity intakes cost more than intakes

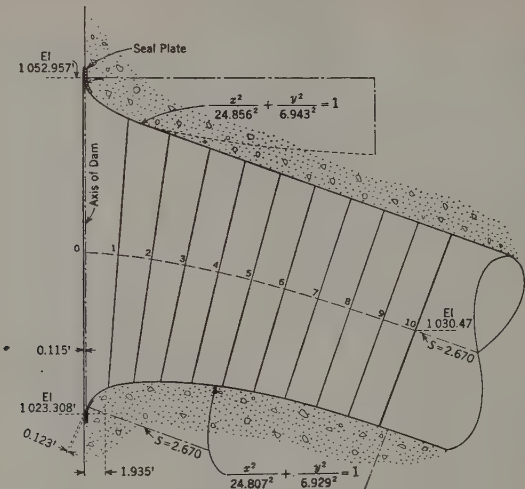


Fig. 10.—Intake Transition of Grand Coulee Penstock

TABLE 1.—Areas and Dimensions of Sections in Transition (See Fig. 10)

Section	Dimensions at Center Line, in Feet		Ellipse Axes, in Feet		Area of Sections, in Square Feet		Developed Distance from the Origin at Seal Plates	
	Height	Width	Vertical	Horizontal	Jet‡	Actual§	Roof‖	Floor¶
0*	29.65	15.00	Rectangle		444.7	444.7	0	0
1	23.72	15.30	11.86	0.63	356.6	356.5	5.45	3.30
2	22.09	15.60	11.05	2.21	324.5	323.7	9.25	5.60
3	20.90	15.90	10.45	3.21	305.1	303.5	12.95	7.95
4	20.05	16.20	10.03	3.99	292.5	290.5	16.10	10.70
5	19.34	16.50	9.67	4.65	284.2	280.5	19.35	13.40
6	18.82	16.80	9.41	5.35	278.2	273.0	22.57	16.10
7	18.48	17.10	9.24	6.18	275.0	267.0	25.77	18.85
8	18.25	17.40	9.13	7.05	273.0	262.3	28.80	21.70
9	18.10	17.70	9.05	8.05	271.9	257.8	31.92	24.60
10	18.00†	18.00†	Circle		271.7	254.5	34.95	27.40

* Gate seal. † Diameter. ‡ Normal to axis of dam. § Normal to flow. ‖ Origin at Elevation 1,052.957. ¶ Origin at Elevation 1,023.308.

designed on the basis outlined herein, but the hydraulic losses may actually be greater due to the formation of eddies and resulting turbulence.

The location of the forebay apron with respect to the lower gate sill shown in Fig. 8 is determined by the velocity formulas presented in 1926 by C. W.

Harris,[3] M. Am. Soc. C. E. Referring to Fig. 11:

$$y = \frac{K'}{x^4} \qquad \text{(2)}$$

In Equation (2), if $C_c = 0.6$,

$$K' = \frac{0.6^2}{4} r^4 H \qquad \text{(3)}$$

and,

$$y = \frac{0.09\, r^4\, H}{x^4} \qquad \text{(4)}$$

in which: r = radius of orifice at the wet face of the plate, and H = the head on the orifice. To orient the curve, let $x^4 = \left(x - \frac{z}{x^4}\right)^4$. By limiting conditions $z = 0.4523\, r^5$. Under conditions of spouting flow through the orifice, the pressure reduction at a distance x from the center line (Fig. 11) is expressed by

$$y = \frac{0.09\, r^4\, H}{x - \left(\frac{0.4523\, r^5}{x^4}\right)^4} \qquad \text{(5)}$$

In terms of velocity and penstock diameter, Equation (5) may be written:

$$v = \frac{0.125\, D^2\, V}{x - \left(\frac{0.507\, D^5}{x^4}\right)^2} \qquad \text{(6)}$$

in which v = velocity at a point distant x from the center line of the penstock (measured parallel to the face); D = diameter of penstock; and V = velocity of the water in the penstock. Tests conducted by the U. S. Bureau of Reclamation indicate that values determined by Equation (6) are slightly greater than those found in a model. For small values of v, as compared to V (that is, $v < 10\%\ V$), the following approximate formula can be used:

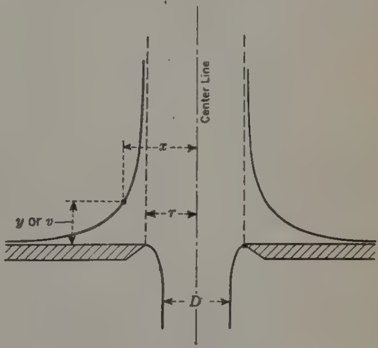


Fig. 11

$$x = 0.3531\, D \left(\frac{V}{v}\right)^{0.5} \qquad \text{(7)}$$

The velocity curve is constructed in accordance with Equation (6) or Equation (7) on the basis of the head on the orifice being the velocity head in the penstock. A point is selected along the face of the inlet for the location of the apron which corresponds to the silting velocity used in the analysis of the intake.

It is assumed that the apron should not be lower than the point where silting will occur, as the rock area below this point will be ineffective. This basis of

[3] "Pressure Reduction on the Face of Orifice Plates and Weirs," by C. W. Harris, *Bulletin No. 35*, Univ. of Washington, 1926.

design has been verified in part by reports of the investigation of the silt level in the intake structures of the sluice-gate chamber at the power plant of the Tennessee Valley Authority at Norris Dam.

The application of basic principles of design to elbows and turns in the penstock is producing passages with a high degree of hydraulic efficiency. By the application of the resolution of velocities to turns in the penstock, it is possible to prevent reflected turbulence inherent when constant-area elbows and turns are used. Approximately streamlined flow is obtained by increasing the velocity around the turns. In most penstock systems, accelerating elbows can be used without obtaining too small a final pipe size, since the pressure head and water-hammer head increase progressively as the length of penstock is increased; and thus the economical diameter of the pipe decreases. A penstock system designed in accordance with this principle is shown in Fig. 9.

In designs where several decreases in size of penstock at turns, as shown in Fig. 9, are not desirable, accelerating elbows may be used by placing an expanding section of conduit immediately downstream from the elbow to maintain a constant diameter of penstock. The so-called inherent shock losses in expanding sections are minimized when the areas of the sections are favorable to the conversion of velocity to pressure head in accordance with the laws of conservation of momentum and pressure. Expanding sections, intended to fulfil the momentum laws, have been designed and tested, and the outflow velocity has been determined experimentally as 98% of the spouting velocity, including friction losses, which undoubtedly account for a greater part of the total loss than the so-called losses of impact.

Gates and Valves

With water power now recognized as a national resource to be developed to its maximum use, it is important that valves be designed for high hydraulic efficiency. Suitable gate valves are being developed in large sizes and for high heads incorporating passages that are straight pipe sections without hydraulic losses. Where butterfly valves appear to be the only practical solution, water passages may be so designed as to result in a loss of head through the valve of not more than 10% of the velocity head in the pipes. Losses through the 120-in. butterfly valves at the power plant at Hoover Dam are shown in Table 2 and Fig. 12. The low losses for this valve are due to the small discharge diameter as compared with the nominal diameter, resulting in accelerated flow that suppresses eddy formation downstream from the valve disk. Usually these losses vary from 20% to 30% of the pipe velocity head when only the mechanical features of the valve are considered.

One of the most important points in gate-valve design has been determined as the hydraulic downthrust on top of the gate leaf when the valve is being closed under spouting velocity conditions. These downward forces have been found to exceed the total weight of the gate leaf by several hundred per cent; thus, a valve leaf hoist designed to lift a load of 100,000 lb would be required to overcome a maximum load of 200,000 lb due to the hydraulic thrust alone. This condition can be relieved greatly by preventing full penstock pressure from

developing on the top of the gate leaf where the valve is placed in the penstock line. This may be accomplished by making close clearances on the upstream face of the valve leaf where high pressures are developed, and liberal clearances on the downstream faces where the low pressure region exists.

TABLE 2.—LOSS THROUGH BUTTERFLY VALVE AT HOOVER DAM POWER PLANT ($M = 135,670$)

Run No.	Deflection*		Discharge in cubic feet per second	Velocity Head		Increase in velocity head through valve	Loss through valve
	Inches of mercury	Feet of water		Down-stream†	Up-stream‡		
1	0	0	4	0	0	0	0
2	0.10	0.10	116	0.10	0.05	0.05	0.05
3	0.30	0.31	234	0.41	0.21	0.20	0.11
4	0.50	0.52	375	1.06	0.55	0.51	0.01
5	1.20	1.26	524	2.07	1.07	1.00	0.26
6	1.90	1.99	677	3.46	1.78	1.68	0.31
7	2.90	3.04	823	5.11	2.63	2.48	0.56
8	3.95	4.14	967	7.06	3.64	3.42	0.72
9	5.05	5.29	1,099	9.12	4.70	4.42	0.87
10	6.09	6.38	1,219	11.22	5.78	5.44	0.94
11	6.81	7.13	1,281	12.39	6.39	6.00	1.13
12	6.80	7.12	1,276	12.30	6.34	5.96	1.16
13	6.60	6.91	1,258	11.95	6.15	5.80	1.11
14	6.40	6.70	1,239	11.60	5.97	5.63	1.07
15	5.80	6.07	1,183	10.58	5.45	5.13	0.94
16	5.60	5.86	1,165	10.25	5.28	4.97	0.89
17	5.30	5.55	1,127	9.60	4.94	4.66	0.89
18	5.05	5.29	1,092	9.01	4.64	4.37	0.92
19	4.80	5.03	1,067	8.60	4.43	4.17	0.86
20	4.50	4.71	1,031	8.02	4.14	3.88	0.83
21	4.15	4.35	1,006	7.64	3.94	3.70	0.65
22	3.90	4.08	967	7.06	3.64	3.42	0.66
23	3.60	3.77	928	6.50	3.35	3.15	0.62
24	3.40	3.56	887	5.94	3.06	2.88	0.68
25	3.10	3.25	859	5.57	2.87	2.70	0.55
26	2.80	2.93	823	5.11	2.63	2.48	0.45
27	2.35	2.46	743	4.17	2.14	2.03	0.43
28	1.90	1.99	669	3.38	1.74	1.64	0.35
29	1.50	1.57	583	2.57	1.32	1.25	0.32
30	1.20	1.26	519	2.04	1.10	0.94	0.32
31	0.85	0.89	449	1.52	0.78	0.74	0.15
32	0.40	0.42	305	0.70	0.36	0.34	0.08
33	0.10	0.10	173	0.23	0.12	0.11	−0.01
34	0	0	4	0	0	0	0
35	0	0	4	0	0	0	0

* 1 in. of mercury $= \frac{13.5670 - 1}{12} = 1.04725$ ft of water. † $h_v = 0.00000755\ Q^2$ at downstream section. ‡ $h_r = 0.00000389\ Q^2$ at upstream section.

The solution of the problem of excessive and prohibitive downpull, due to the hydraulic thrust on penstock gates placed at the penstock entrance, as shown in Fig. 8, is to shape the bottom to the gate so as to maintain the highest hydrostatic pressure possible on the bottom of the gate under critical flow conditions, and lower the bottom of the intake flume an amount to give an estimated velocity of not more than 2 ft per sec under the most severe flow conditions. This basis of design results in a penstock gate with a false bottom of light construction so shaped as to give favorable uplift pressures under emergency closing conditions. The bottom seal is placed on the downstream face of the gate similar to the side and top seals.

By analyzing pressures on the gate for various positions and under different rates of flow, it has been possible to reduce the cost of the gate hoists a sub-

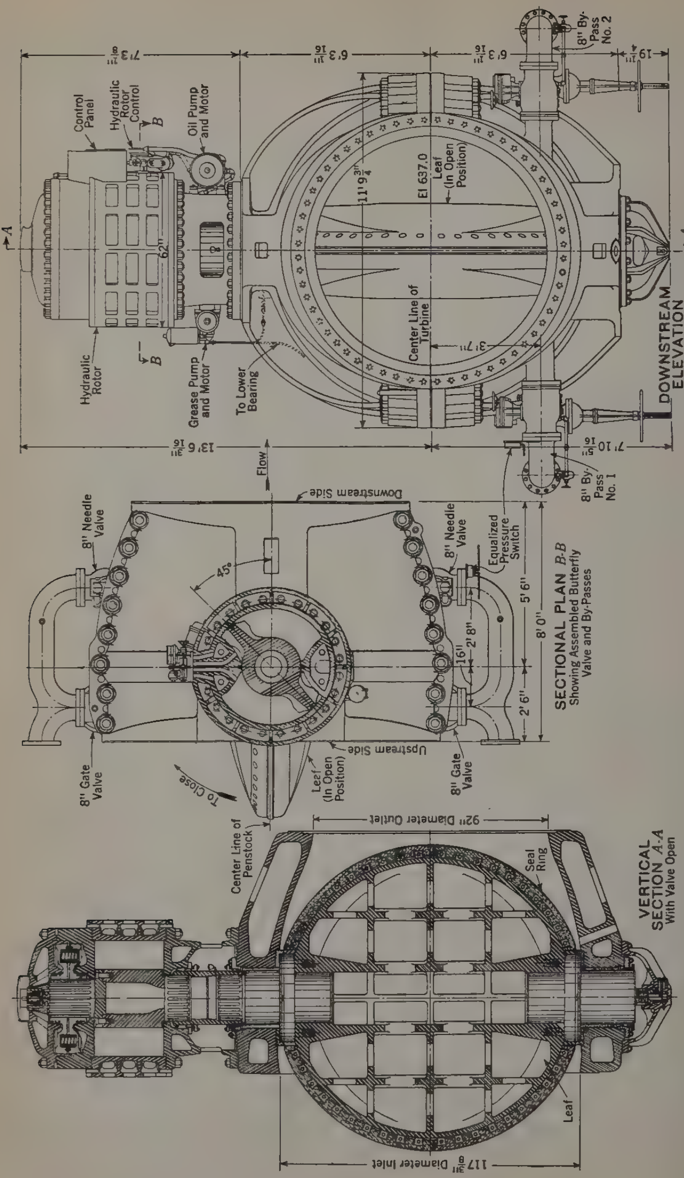


FIG. 12.—BUTTERFLY VALVE (120-IN.) AT HOOVER DAM (SEE TABLE 2 FOR THE VELOCITY HEAD LOSS FOR VARIOUS RATES OF FLOW)

stantial amount and to increase the hydraulic efficiency of the intake. In addition to these advantages, trouble due to debris collecting in the recess slot at the bottom seal has been eliminated. By the use of a vertical face at the gate entrance for a distance of from 6 ft to 7 ft below the intake, it is unlikely that any form of material will collect on the lower seats. All of this fits into the general scheme of developing intakes that utilize higher velocities, obtain better efficiency, and are constructed at lower cost.

Scroll Cases

It is believed that there are possibilities in the development of scroll cases with appreciably higher velocities than those normally employed which is in agreement with the results obtained in the series of pump tests previously mentioned. Such cases may not require movable gates or guide vanes, the power of the unit being controlled by a single valve placed at the entrance of the casing similar to the Reiffenstein turbine.[4] In the present stage of development the single-gate turbine is suitable principally for small units, as the control features are inferior to the present type. Furthermore, the unbalanced side thrust from the runner is high, requiring a large shaft. The relatively large field clearances and the tangential velocity component are too high for use in connection with the propeller turbine. Rapid progress is reported toward a solution of these difficulties by the study of the characteristics of the forced spiral vortex and the application of these principles in the design.

Several low-head plants have been built with improved scroll cases, including those at Wheeler Dam, Bonneville Dam, Drop No. 4, and other plants. The casings at Wheeler Dam and Drop No. 4 have three penstock intakes coming into one scroll-case chamber, the water seeking its direction into the casing as required. Sectional plan of the Drop No. 4 casing is shown in Fig. 13. The Bonneville casing employs an island to form two separate channels in one third of the casing. Results of field and laboratory tests indicate that the distribution in each of the three passages at Bonneville was substantially equal. The distribution at Wheeler and Drop No. 4 varied between 30% and 40% for the three separate penstocks. The development work on these designs consistently indicated that better guiding of the water at increased velocity tended to produce better performance. This fact was demonstrated at Wheeler Dam by an increase in horsepower of the turbine throughout the full range of gate opening when the areas in the small part of the casings were restricted by contracting the floor and roof of the casing to produce better guiding of the water into the turbine.

As an example of the results of careful attention to basic hydraulic principles in design, it was possible to remove the trashrack structures, complete intake with penstock, and the scroll case from the model turbine used in the acceptance tests of the Drop No. 4 turbine, and not lose any over-all efficiency or horsepower capacity of the turbine. In fact the detail characteristics of the gate-opening speed curves were slightly better for the test when the power-

[4] *Wasserkraft und Wasserwirtschaft*, Heft 11–12 (33, Jahrig, 1938), pp. 144 and 145.

plant conduits were in place than were those obtained for the open-flume conditions. The reason for these apparently inconsistent results is obviously better distribution of flow of water into the turbine guide case with the scroll case in place than with the turbine tested under open-flume conditions. Thus, the

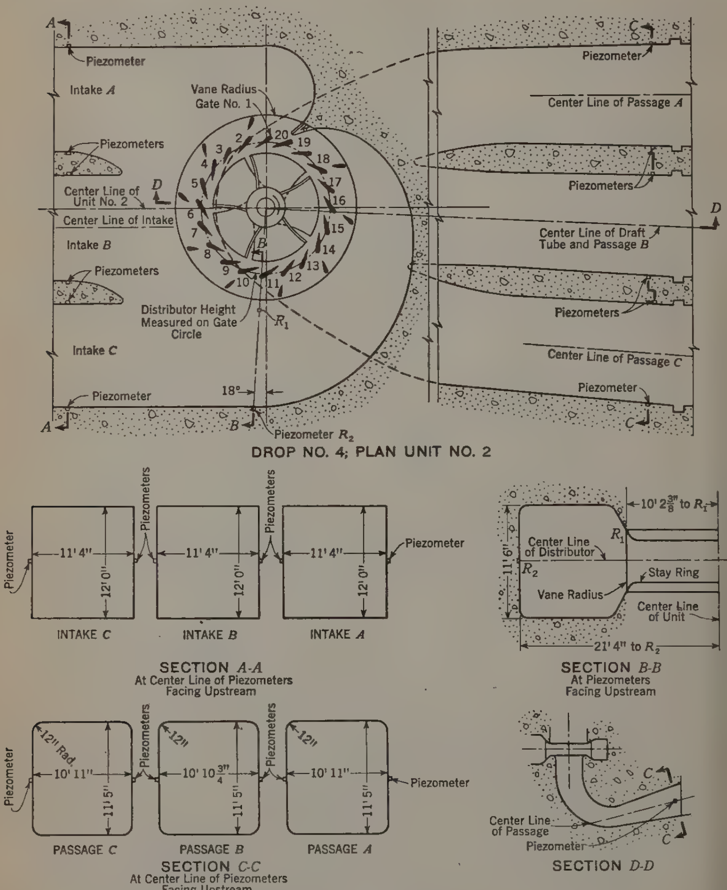


FIG. 13.—SCROLL CASE AND DRAFT TUBE OF DROP NO. 4 POWER PLANT

method of designing hydraulic passages by assuming that large areas result in low velocities and improved hydraulic efficiency may lead to inadequate designs. Most rules of this nature have been devised because of the timidity of the engineer in approaching hydraulic designs involving turbulent flows.

Draft Tubes

The trend of draft-tube design in recent years has been toward the elbow, as the concentric type of tube has proved to be expensive, with no advantage in efficiency or power over the elbow tube. A strong tendency is now apparent for American engineers to take up the problem of elbow draft-tube design incorporating the horizontal splitter, although the European engineers who developed this type of tube are discarding it for the plain elbow. This change in opinion may be the result of the attempt to lower excavation costs in America and the attempt to obtain better efficiency in the European installations. The energy of the engineers in the United States will be directed toward both of these objectives as more of the characteristics of the splitters are better understood.

Experimental Investigations

In low-head, large-volume turbine installations, where field measurements of water are exceedingly difficult, acceptance of performance is being based on laboratory tests of strictly homologous model units, incorporating, as completely as possible, all water passages of the prototype that affect the performance of the turbine. This method of turbine acceptance offers advantages over a field test in the respect that if the tests indicate that changes in design are desirable, the necessary development work can be performed and incorporated in the final design. Where desirable changes in design are discovered during field tests on the prototype, it is usually too expensive, or it is impracticable, to make such changes in the plant.

As an alternate to model tests in the laboratory, it is sometimes feasible to base the turbine acceptance tests on a small station-service unit which is similar to the main unit. In effect, this is the use of a model of intermediate size and offers the advantage of both the field test and model test; but it is not favorable from the standpoint of development work during tests.

The field performance of a turbine that has been accepted on the basis of laboratory tests on homologous units can be estimated by suitable power step-up methods which evaluate the increase or decrease in horsepower between model and prototype, as a measure of the efficiency of the field unit. Calibration of the deflection of the differential pressure taps, placed in the model of a scroll case, as related to the flow through the turbine in cubic feet per second, can be used to estimate the flow in the prototype by placing similar piezometer connections in the scroll case.

Fig. 13 represents a typical installation of piezometer taps designed to coordinate pressure in the penstock, scroll case, and draft tube for the comparison of model and prototype hydraulic performance. By placing a V-tube manometer across the pressure taps in the lower section of the penstock at section *AA* and at the discharge orifice of the draft tube at section *CC* the true net head acting on the turbine is obtained and also the coefficient of the turbine as a meter. A second V-tube manometer to measure the differential pressure in the scroll case designated as R1–R2 at section *BB* affords a second independent comparison of flow in the model and prototype. Thus it is possible to obtain a comparison of power and discharge between model and prototype.

At the U. S. Engineers' hydraulic laboratory at Portland, Ore., studies of draft-tube shapes were made with pyralin models constructed to a scale of 1 : 46—that is, with a 6-in. throat diameter, first alone and later with model intakes and runners. Several alternatives of the elbow type were studied. The most valuable result from these experiments was the establishment of the fact that a horizontal spreader improved the stability and the efficiency of all draft tubes studied, both alone and with scroll and runner attached. These experiments were further verified at the laboratory of the turbine manufacturer, using an homologous turbine runner 16 in. in diameter.

Extensive model tests of different types of intakes on a 1 : 46 scale were made, using wood with pyralin windows. Visual methods of analysis were used with ribbons and sawdust to show the direction of flow. Piezometers were placed in the different intake passages to obtain equal head losses and equal water distribution among the passages. Motion pictures were taken and these were studied to improve the flow conditions within the water passages. By these means of investigation, very close to equal distribution of flow around the speed ring was obtained. The tests and studies resulted in the selection of an intake with long division piers between the water channels and with islands in one of the three channels to distribute the water proportionately around the wheel casings. A constant velocity scroll gave the best results, although both accelerating and decelerating scrolls were investigated.

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MODEL AND PROTOTYPE TESTS

By L. M. Davis,[5] Assoc. M. Am. Soc. C. E.

Synopsis

The development of the methods of testing turbine models is described in this paper. The writer deals particularly with the more recent progress that has been made in determining cavitation characteristics. The interpretation of the laboratory results are important not only for designing the proper setting of the prototype to avoid cavitation, but also for predicting the power that can be expected. The application of the Moody formula for this purpose is discussed. Three outstanding methods used in the United States for measuring the quantity of water taken by even the largest hydraulic units can be depended upon for reliable results, and failure to make acceptance tests may retard progress materially in the design of future installations. In conclusion the collection of data by which the step-up formulas can be verified is emphasized as being of utmost importance.

Introduction

Water wheels were in use for many centuries with only a few improvements on the original crude designs. It was not until 1827 that the modern turbine was invented by Fourneyron. About twenty years later the Francis turbine was developed by means of careful laboratory tests conducted by the late James B. Francis,[6] Hon. M. Am. Soc. C. E. By the beginning of the twentieth century, Francis turbines were available for heads up to several hundred feet, with efficiencies of about 80%. The increasing popularity of electric power tremendously increased the demand for turbines. Numerous concerns went into the business of manufacturing turbines, and many of them built turbine testing laboratories as a means of improving their product to meet competition. By 1915 the art of designing Francis turbines had advanced so that efficiencies in excess of 90% were common.

Since that date the improvement in turbine efficiency has been slight, but efficient designs have been developed for heads up to 1,000 ft. Development of the impulse wheel has followed that of the Francis turbine so that their efficiency is now only slightly less than 90%.

In the early days of turbine design it was considered impossible to have the peripheral speed of an efficient turbine exceed the spouting velocity of the water. Low heads resulted in low spouting velocities, which limited the peripheral speed. Since an increase in runner diameter meant a reduction in rotative speed and an increase in generator size for the same peripheral speed,

[5] Engr., Pennsylvania Water & Power Co., Holtwood, Pa.

[6] *Encyclopedia Britanica*, 14th Edition, Vol. 22, p. 580.

it was common practice to connect several small turbines to one generator to obtain the desired power without sacrificing rotative speed. In some instances as many as six turbines were connected to one generator.

It was soon discovered that the ratio of peripheral velocity of the turbine to the spouting velocity of the water (a ratio known to hydraulic engineers as ϕ) could exceed unity without a sacrifice in efficiency.

Development of Propeller Turbine

In order to reduce generator costs to a minimum, the turbine manufacturers attempted to design high-speed turbines for low-head developments. This trend led to the introduction of the propeller turbine, which has a high specific speed.

Specific speed may be defined as the revolutions per minute at which a runner would rotate if it were so reduced in proportion that it would develop 1 hp under a 1-ft head. In runners with high specific speeds the relative velocity between the water and runner is greater than for runners with low specific speeds. It is evident from Fig. 14 that in attaining high specific speeds there was a considerable sacrifice in efficiency at part loads.

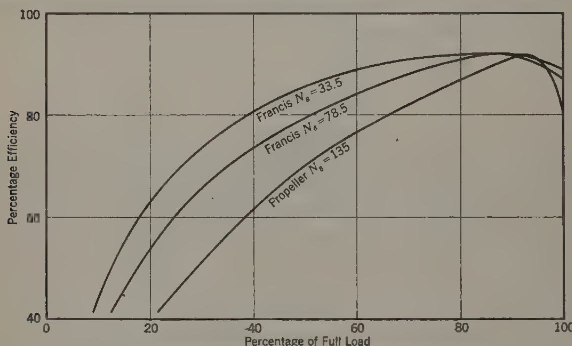

Fig. 14.—Effect of Specific Speed on Turbine Efficiency

From laboratory tests Prof. Viktor Kaplan found that, by reducing the blade angle of a propeller turbine at part load, the efficiencies could be improved greatly, and that by increasing the blade angle the capacity could be increased greatly without much sacrifice in efficiency. Consequently, in 1919, he developed what is now known as the Kaplan turbine in which the blade angle is automatically adjusted with load changes to have a predetermined relation to the gate opening, resulting in a flat efficiency curve over most of the operating range, as shown in Fig. 15.

Since about 1914 a turbine testing laboratory capable of determining the efficiency characteristics of carefully made models of turbines with their settings, draft tube, intake, and scroll case has been considered an indispensable part of a turbine manufacturer's equipment. It seems unnecessary in this

paper to include a description of such a laboratory. In 1938 Mr. L. J. Hooper gave an excellent description[7] of most of the turbine testing laboratories in the United States.

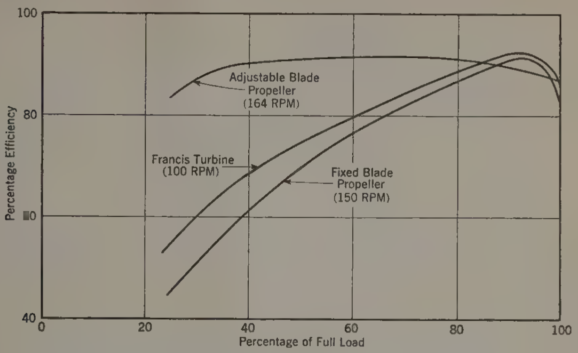

FIG. 15.—COMPARISON OF EFFICIENCY CURVES OF THREE 9,500 HP TURBINES AT 38-FT HEAD

CAVITATION AS A VITAL PROBLEM

Increasing the specific speed of turbines led to very high relative velocities between the turbine blades and the water, as well as high actual velocities, with a corresponding conversion of pressure head into velocity, which in turn resulted in low pressures in the turbine. This condition has been conducive to a phenomenon known as cavitation, which has become a vital problem.[8]

Cavitation is not new to hydraulic engineers. Some of the early Francis turbines were subject to serious cavitation caused by faulty design or to locating the turbine too far above the tailwater.

As the name implies, cavitation involves the formation of cavities in the liquid. These cavities are made possible when the pressure is reduced to the vapor tension. When this occurs, the liquid "cold boils," forming vapor which fills the cavity. The cavities are carried along with the flowing water; and when the pressure increases above the vapor pressure, the vapor turns to liquid and the cavities suddenly collapse. This violent collapse creates high water-hammer pressures concentrated on a very limited area. Striking upon the boundary walls, these blows are undoubtedly repeated at a high frequency, finally resulting in a failure of the boundary material known as pitting.

The possibility that these blows are severe may be appreciated if one considers that the collapse of a spherical void in an inelastic medium, theoretically, will cause an infinite pressure at the moment of complete collapse. The elasticity of the water imposes very definite limitations which are still further reduced by the presence of non-condensable gases such as air. Con-

[7] "Representative Hydraulic Laboratories in the United States and Canada," by L. J. Hooper, *Journal*, Boston Soc. of Civ. Engrs., January, 1938.

[8] "Vortrag auf der Weltkroftkonferenz," by D. Thoma, Germany, 1924; also "Investigations into Causes of Corrosion or Erosion of Propellers," by Sir Charles A. Parsons and Stanley S. Cook, *Engineering*, April 18, 1919.

sidering the elasticity of water, it is difficult to believe that the fatigue limits of the materials used can be exceeded. The fact that pitting actually can be produced in any material so far studied leads to the supposition that the pressure is generated not by the simple falling in of the walls of a sphere, but is augmented by some accumulative effect. W. Watters Pagon,[9] M. Am. Soc. C. E., has shown that the most likely form of the cavity is the core of a vortex, the progressive collapse of which can give rise to the projection of particles of water at terrific velocities.

As might be expected, cavitation has an adverse effect on the efficiency of a turbine. The power output is reduced below normal and the discharge is increased.

Cavitation Laboratories

In order to study the cavitation characteristics of a model turbine, it is necessary to subject it to the same pressure conditions that will be encountered in the prototype. This requirement called for a new type of turbine testing laboratory.

A cavitation testing laboratory for model turbines is similar to the usual turbine testing laboratory, except for the wider variation of the headwater and tailwater levels and for the higher test heads. In order to reproduce all possible head conditions to which a turbine may be subjected, it is necessary to be able to vary the headwater and tailwater levels over a wide range, thus varying the draft head and the pressure conditions on the turbine. When first attacking the problems of determining the cavitation characteristics of a turbine design, it appeared necessary to test the model under the same head conditions which prevail in the field. Experience has now shown that this is not necessary. In the case of the Holtwood (Pa.) laboratory where cavitation data were taken for heads varying from 45 ft to 58 ft, there was no noticeable effect due to head variation. However, Prof. W. Spannhake conducted similar tests in the cavitation laboratory at Karlsruhe, Germany, under heads from 1 m to 10 m (3.3 ft to 32.8 ft) and found a decided influence of head at the lower values. This effect seemed to be completely "ironed out" by the time the 10-m head was reached. Of course, in order to obtain the desired draft head when the total head is low, it is necessary to reduce the pressure on the entire system.

Since several headwater and tailwater combinations result in the same pressure conditions in the runner, it is essential to have a cavitation coefficient which will indicate the true pressure conditions on the runner for any possible combinations. Through his outstanding work[10] at Munich, Germany, Prof. D. Thoma originated the commonly accepted cavitation coefficient, Σ. The formula for Σ is:

$$\Sigma = \frac{H_b - H_s - H_v}{H_t} \qquad (8)$$

in which H_b = barometric pressure in feet of water; H_s = draft head, in feet; H_v = aqueous vapor pressure, in feet of water; and H_t = total head, in feet.

[9] "Cavitation and Erosion Investigated as a Problem in Fuid Mechanics," by W. W. Pagon, presented at the Annual Meeting of the A. S. M. E., 1935.

[10] Described in a Talk by Prof. Thoma at the Fall Meeting of the Society, October 10, 1929. For a brief abstract of the Paper see *Proceedings*, Am. Soc. C. E., December, 1929, p. 2561.

The first cavitation testing laboratory for turbine models in the United States was built in 1930 by the Pennsylvania Water and Power Company at Holtwood for use in designing the 42,000 hp Kaplan turbines for the Safe Harbor Development. The Shawinigan Water and Power Company of Montreal, Que., Canada, built their cavitation turbine testing laboratory prior to 1930, but it was in use on an extended testing program during the time the Safe Harbor tests were conducted. There were also certain head limitations which did not fulfil the requirements for the Safe Harbor tests. The layout of the Holtwood laboratory and the procedure of conducting cavitation tests was described in two papers published[11] in 1934 and 1935.

There are three other laboratories in the United States capable of determining the cavitation characteristics of a model turbine. The test procedure is essentially the same in all of them. Three of the four laboratories are equipped to test models of the complete unit, including intake passages, scroll case, wheel setting, turbine, and draft tube. A complete model of all water passages is usually considered highly desirable since disturbances from upstream of the turbine may have an effect on the cavitation characteristics.

In the early days of cavitation testing of model turbines there was considerable controversy as to the best means of determining the point at which cavitation begins as the draft head is progressively increased. Most early laboratories were equipped with glass windows in the draft tube for visual observation. Although it is extremely interesting to observe the turbine under various operating conditions, the visual method of determining the starting point of cavitation was found to be unreliable. Not only would two people place a different interpretation on what they observed, but it is impossible for the same person to compare visual observations accurately from day to day.

At present the commonly accepted method of determining the start of cavitation is by quantitative measurement, making use of the fact that cavitation reduces power and increases discharge, with a resulting drop in efficiency. Fig. 16(*a*) shows the data obtained from a cavitation test at three gate openings and at a constant value of ϕ. This test was at one blade setting of a model of a Kaplan turbine. It will be noted that efficiency, unit horsepower, and unit discharge are all plotted against Σ.

It should be stated that Σ is used in two ways: (1) In reference to the plant and (2) in reference to the turbine. Plant sigma is the effective sigma as computed for the installed unit. The value of plant sigma will vary with headwater and tailwater elevations, barometric pressure, and vapor pressure of the water, which in turn varies with water temperature. Critical sigma is the value at which cavitation begins in a turbine for a given set of conditions of gate opening, blade angle (for Kaplan or adjustable blade propeller turbines), speed, and head. The purpose of the cavitation laboratory is to determine the critical sigma values for various operating conditions. It follows logically that if the plant sigma should be less, at any time, than the critical sigma, cavitation could be expected.

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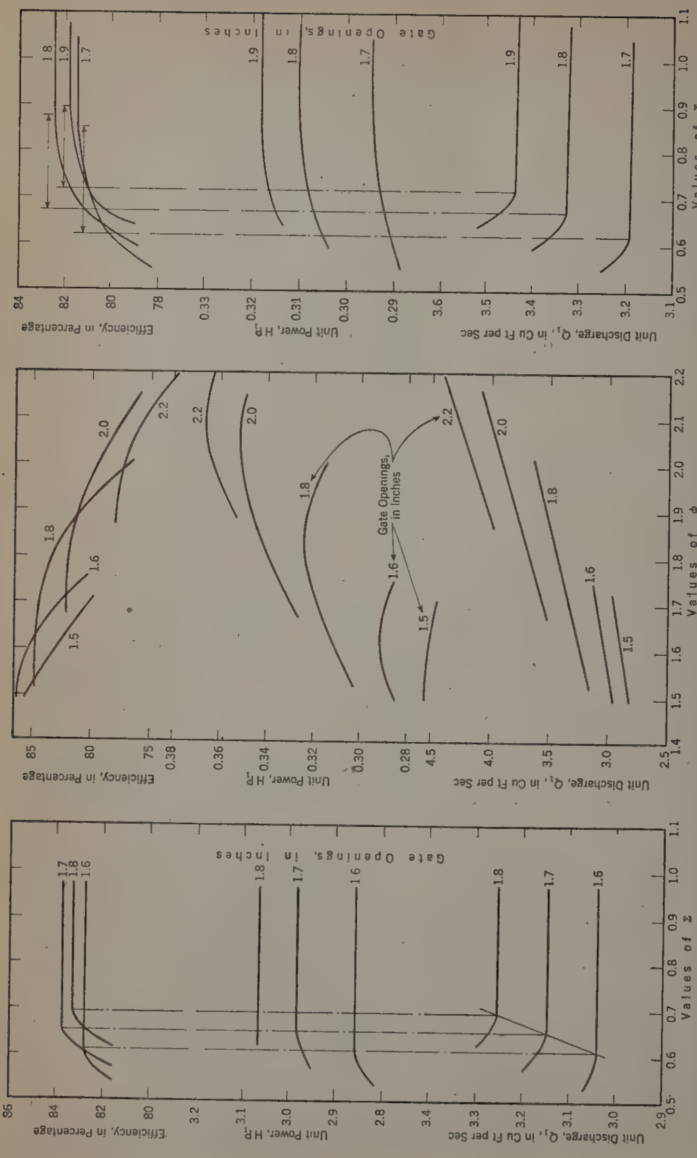
[11] "Model Testing at Holtwood Hydraulic Laboratory," published by the Pennsylvania Water and Power Company, May, 1934; and, "Cavitation Testing of Model Hydraulic Turbines and Its Bearing on Design and Operation," *Transactions*, A. S. M. E., November, 1935.



(*a*) Cavitation Test (Σ); Properly Designed Propeller; Runner 2.1; $\phi = 1.635$

(*b*) Efficiency Test (ϕ); Runner 2.1; Head, 36 Ft; $\Sigma = 1.22$; Alternate Gate Curves Omitted

(*c*) Cavitation Test (Σ); Poorly Designed Propeller; Runner 1.2; $\phi = 1.686$

FIG. 16.—TYPICAL RESULTS ON A PROPELLER TURBINE (BLADE ANGLE, 26°; GATE POSITIONS AS SHOWN)

To determine the cavitation characteristics of a fixed-blade propeller turbine, it is necessary to run tests at three or four values of ϕ covering the operating range of heads and for various gate settings from the maximum down to a low enough gate setting so that the critical sigma is well below the lowest value of plant sigma. In the case of a Kaplan or adjustable-blade propeller turbine, this procedure would require much unnecessary work since such a turbine is operated so that the most efficient relation between gate opening and blade angle is maintained at all times. In order to secure the information in the operating range only, efficiency tests are made over the entire operating range of ϕ-values for each blade setting of the model (see Fig. 16(*b*)). From these data, the three gate settings which bracket the peak efficiency at the value of ϕ in question are determined and cavitation tests are made on them as shown in Fig. 16(*a*). It is the practice in the Holtwood laboratory to use the break in the unit-discharge-sigma curve in arriving at the critical sigma. If the efficiency and unit-power curves break at a higher value of sigma than the unit-discharge curve, this higher value is used with the unit discharge obtained at that sigma. Cavitation tests are made at three or four values of ϕ so that the data may be presented as shown in Fig. 17. It is an easy matter to cross-plot and locate intermediate ϕ-curves.

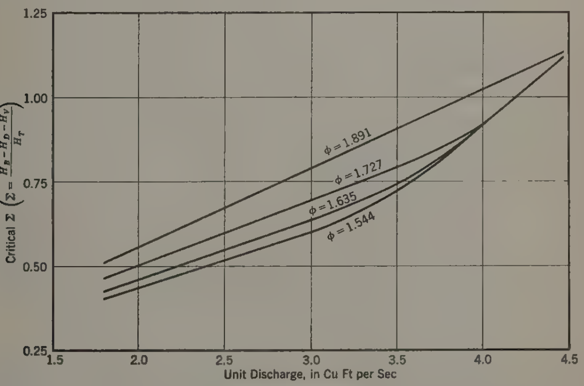


FIG. 17.—CRITICAL VALUES OF Σ VERSUS UNIT DISCHARGE FOR VARIOUS VALUES OF ϕ

Applying the data in Fig. 17 to a specific development, it is possible to determine the proper elevation of the turbine with reference to tailwater levels and also to determine the safe load that can be carried at heads other than the rated head. In arriving at the proper turbine setting after the turbine diameter, rotative speed, and maximum output at rated head have been decided upon, it is only necessary to determine the unit discharge corresponding to the maximum output at rated head and apply this value in Fig. 17 to obtain the critical Σ-value for the value of ϕ corresponding to rated head. Using this value of critical sigma in the sigma formula, the allowable draft head may

be computed. It is common practice among turbine manufacturers to insist that the turbine be set a certain distance lower than indicated by this computation (1.5 ft in the case of Safe Harbor) as a margin of safety.

After the turbine setting has been determined, the plant sigma values can be computed for various headwater and tailwater conditions; and, by the use of Fig. 17, the allowable unit discharge is obtained from which the allowable power output is computed. For the convenience of operators, these data can be presented in a chart of constant power lines with headwater elevations as abscissas and tailwater elevations as ordinates.

In most developments it is of economic importance to locate the runner at as high an elevation as possible so as to keep the excavation cost at a minimum. Therefore, some idea of the importance of an accurate determination of the critical sigma of a turbine may be understood when it is realized that a variation in draft head of 1 ft at Safe Harbor affects the allowable power output 3.5% at normal head.

Fig. 16(*c*) presents data similar to those in Fig. 16(*a*), but for an entirely different turbine model. It will be noted that in Fig. 16(*c*) there is a wide discrepancy between the value of critical sigma, as indicated by the break in the efficiency curves and that indicated by the break in the unit-discharge curves, whereas in Fig. 16(*a*) the breaks occurred at the same sigma. Undoubtedly, the break in the efficiency curves indicates some cavitation disturbances which do not become sufficiently severe to affect the discharge until a lower sigma is reached. It is the consensus of opinion among hydraulic engineers that failure of the efficiency and unit-discharge breaks to coincide is an indication of faulty design at some point which induces local cavitation to occur ahead of general cavitation.

General cavitation may be defined as the result of the general lowering of pressure in the turbine until the pressure at some point, or over a certain area of each turbine blade, is equal to the vapor pressure. With such reduction, general cavitation is unavoidable regardless of how well designed a turbine may be. However, with cavitation test data at hand, it is a simple matter for the power-house designer to so locate his turbine with respect to tailwater that the possibility of general cavitation will be eliminated. On the other hand, local cavitation is the fault of poor design, and is likely to cause serious pitting even when the general pressures in the turbine are well above the danger limit. There are numerous causes for local cavitation, such as too abrupt curvature in the blade surfaces, reversals in curvature in the direction of flow, roughness or small obstructions, and excessive runner clearances. Local cavitation may also be caused by disturbances of the turbine from upstream such as might be set up by poorly designed wicket gates, gates that overhang the throat ring at or near full opening, poorly shaped stay vanes, stay vanes at improper angles with the flow, and poor design of curb ring, head cover, and scroll case. General cavitation of a turbine is affected by blade area, hub diameter and shape, and draft-tube design. It is evident that the more efficient the draft tube, the lower the general pressure on the turbine will become for a fixed set of head and discharge conditions.

Present knowledge of hydraulic theory and mathematical analysis has proved a valuable aid, but it has its limitations, and the refinements of design must be attained by a "cut-and-try" process in the laboratory. The location of the causes of local cavitation in the cavitation laboratory is extremely difficult and costly since there are relatively few means for detecting its presence. Some work has been done with stroboscope, photography, spark photography, and stereoscopic photography; but all of these methods require clear visibility of the parts affected.

Much valuable information on local cavitation has been obtained by means of paint tests. For such a test the runner is painted and then the test is made before the paint has completely hardened. Where cavitation occurs, the paint is removed. This has been confirmed by experiments where actual pitting on the prototype has been simulated by means of paint tests. This is a rather delicate test to make, since, if the paint is too hard, it will require a long period of running to show the effects of cavitation, and if it is too soft, the high-water velocities may remove the paint by erosion.

The Corps of Engineers, U. S. Army, used models made of a transparent material for making intake, scroll case, and wheel-setting tests for the Bonneville Development. Sawdust with the same specific gravity as water was introduced into the flow, and from slow-motion movies it was possible to trace out the flow lines and detect the presence of eddies. No doubt this method of testing would lead to a quick solution of many of the causes of local cavitation. However, such tests must be made in conjunction with efficiency and cavitation tests. In some cases guiding vanes may seem necessary to remove a disturbance from the flow, but efficiency tests are likely to show that the surface friction losses of the guides are far greater than the gain from eliminating the eddy. Experience seems to indicate that the most efficient intake and scroll case is the one with the fewest piers and obstructions.

As is often the case in a new field of research, certain unjustified liberties were taken in some of the early model tests. At one time it was customary to conduct tests on a model turbine in an open-flume setting with a straight conical draft tube and then attempt to predict the behavior of the prototype having an intake, scroll case, and elbow draft tube. No doubt this practice is responsible for many of the disappointing efficiency results that have been obtained by field tests. Present practice for important installations requires model tests on precise models of the entire water passages from the intake to the draft tube except where the distance from intake to scroll case is great.

Efficiency and Power Step-Up

After the efficiency characteristics of a turbine model have been determined by model tests, it is desirable to be able to predict the characteristics of the prototype. Ordinarily, the percentage of losses in a turbine varies inversely with the size. Thus, in going from model power and efficiency to prototype power and efficiency, an increase in excess of that computed from the theoretical step-up, due to size, can be expected.

There are a number of variable factors which affect the step-up, such as surface roughness of the model as compared to prototype, and the mechanical losses of the laboratory set-up compared to the prototype. For this reason it would seem logical that one step-up relation would not hold exactly true for all laboratories or for all models in the same laboratory. Most models at present are carefully made of bronze and considerable care is taken to give them exceptionally smooth surfaces; so, as a rule, most models have comparable surface roughness.

Since the Holtwood laboratory has been in operation, it has been possible to make direct comparisons with efficiency data taken in two other laboratories. In one case the agreement was within 0.2% over the entire range and in the other case the peak efficiencies agreed, but there was a slight shift in the curves. The usual test head for the Holtwood laboratory is from 35 ft to 50 ft, whereas the other two laboratories tested at 3-ft to 4-ft head (I. P. Morris) and at 8-ft to 10-ft head (S. Morgan Smith Co.), respectively. Because of the wide range of testing heads, the three laboratories must necessarily have radically different test equipment. Yet, in spite of this, one step-up formula should give good results for the three laboratories, for all practical purposes, since the head factor is usually negligible.

The most widely known step-up formula[12] in the United States is one developed about 1925 by Prof. L. F. Moody. This formula is based on a theoretical analysis of the reduction in losses with increase in size. The Moody step-up formula is:

$$e_p = 100 - (100 - e_m)\left(\frac{D_m}{D_p}\right)^{0.25}\left(\frac{H_m}{H_p}\right)^{0.01} \qquad (9)$$

in which: e_p = expected efficiency of prototype; e_m = model efficiency; D_p = diameter of prototype turbine, in inches; D_m = diameter of model turbine, in inches; H_p = prototype head, in feet; and H_m = model head, in feet. Equation (9) applies only to the peak efficiency of the model even if it occurs at a value of ϕ outside of the operating range. To obtain the expected efficiency at off-peak points, the increment of efficiency obtained at the peak efficiency point is added to the model efficiency at those other points.

When applied to complete homologous models, the Moody formula gives results which agree well with field tests for small and medium diameters. From various experience curves that have been developed, it appears that, for large-size turbines, the step-up is somewhat less than that obtained by the Moody formula. Few of the large propeller-type turbines that have been installed in the United States recently have been tested in the field. This is extremely unfortunate since failure to know what actual field efficiencies have been attained may delay advances in design.

In an analysis of step-up of turbine efficiency due to size, W. S. Pardoe, M. Am. Soc. C. E., found that the exponent applied to the ratio of diameters, theoretically, should be 0.20 instead of 0.25. If the Moody formula is modified according to Professor Pardoe's analysis, much better agreement is obtained

[12] "The Propeller Type Turbine," by L. F. Moody, *Proceedings*, Am. Soc. C. E., August, 1925, p. 1009.

between the stepped-up efficiency and the field efficiency for the larger-diameter installations that have been tested in the field. From the data on hand it appears that for the smaller diameters (say, less than 150 in., for which most of the comparative data exist) the exponent of 0.25 gives closer agreement with the field tests than the exponent of 0.20. This tendency might be explained by the fact that all results where the field test reached or exceeded the expected efficiency were used in the experience curve, whereas results that fell short of the expected efficiency were discounted and not plotted.

The Moody formula gives only the efficiency step-up without any attempt to determine what step-up there may be in power. In discussing the subject of power step-up in 1936, F. H. Rogers,[13] assistant chief engineer, Baldwin Southwark Corporation, made a very logical suggestion. It was his opinion that the increase in efficiency of the prototype is the result of a decrease in the internal loss in head, due primarily to the larger water passages. On this basis the increase in efficiency of the prototype is due to a greater effective head, causing an increase in the quantity of water but a larger increase in the power output.

To determine the relative increase in power output as compared to water quantity, let h = head on model, Δh = gain in head on prototype, e_m, Q_m, P_m = efficiency, water quantity, and power output for model, and e_p, Q_p, P_p = efficiency, water quantity, and power output for prototype. Then:

$$e_p = e_m \frac{h + \Delta h}{h} \qquad \text{(10}a\text{)}$$

$$Q_p = Q_m {\frac{h + \Delta h}{h}}^{0.50} \qquad \text{(10}b\text{)}$$

and,

$$P_p = P_m {\frac{h + \Delta h}{h}}^{1.50} \qquad \text{(10}c\text{)}$$

For the usual small values of Δh this means that the percentage increase in power output will be about three times the percentage increase in the quantity of water and about one and one half times the increase in efficiency.

An analysis of a large number of comparative model and prototype tests on Francis turbines showed that, although the efficiency step-up agreed reasonably well with the Moody formula (Equation (9)) when the turbines were truly homologous as far as could be ascertained, the power varied from a negative 6% to a positive 10% from the value obtained by multiplying the model power by the ratio of heads to the three-halves power and the ratio of diameters squared.

From the rather meager comparative data available on propeller-type turbines, there seems to be a definite step-up in power which agrees with the value predicted by Mr. Rogers.

Turbine designers agree that in all probability failure to get a set-up in power is due to the fact that the prototype is not exactly homologous to the model. The form of the propeller turbine is much simpler than the Francis

[13] *Transactions*, A. S. M. E., May, 1936, p. 327.

turbine, and it is, therefore, much easier to make the model and prototype homologous.

It appears that, for propeller-type turbines, the expected power can be predicted accurately, but for Francis turbines extreme caution must be exercised to insure that the model and prototype are homologous before an accurate prediction can be expected.

Field Tests

As the demand for hydroelectric power increased, it became increasingly more important to use the available water at a given development in the most efficient manner. In certain developments, also, governmental requirements called for an accurate accounting of the water used. Consequently, it became vitally important to determine the actual efficiencies of the units. The electrical output could be measured well within 1% and, by suitable tests, the mechanical and electrical losses of the generator, turbine, and bearings could be determined accurately, giving a value of turbine output within 1% of the true value. However, until comparatively recent times, the difficulties of measuring large volumes of water within 1% seemed almost insurmountable.

Early tests with pitot tubes and the salt-titration method gave reasonable results, but both of these methods become unwieldy when the size of the conduit passes medium size and for medium or large volumes of water. The salt-velocity method[14] and the pressure-time method[15] of measuring the flow of water were introduced in 1923. Both of these methods were checked carefully against laboratory standards and were found to give accurate results. These methods are widely used in North and South America and, for suitable conditions, they give results closer than 1%.

Since it depends on the phenomenon of water hammer, the pressure-time method can be applied only to the flow of water in closed conduits. The salt-velocity method can be used in both open channels and closed conduits. Both methods used a section of conduit as a test section, and it is essential that the cross-sectional areas and lengths of the section be known accurately. It is also essential that there be no "dead water" in the test section. A number of important low-head plants have been installed with extremely short intake passages. In some of these developments it is impossible to get sufficient length of water passage to obtain accurate results by either method.

When the Safe Harbor Development was started, it was considered that none of the accepted methods of water measurement in use in the United States would give accurate results; consequently, a survey of methods in use in Europe was made. It was found that a multiple-current-meter method, using meters of the axial flow type,[16] was used almost entirely.

The general procedure followed is to install a frame which travels in the gate slots of the intake. A sufficient number of meters are mounted along a horizontal streamlined member of this frame to insure an accurate horizontal

[14] "The Salt Velocity Method of Water Measurement," by C. M. Allen and E. A. Taylor, Members Am. Soc. C. E., presented at the Annual Meeting of the A. S. M. E., December 3–6, 1923.

[15] "The Gibson Method and Apparatus for Measuring the Flow of Water in Closed Conduits," by N. R. Gibson, M. Am. Soc. C. E., Paper No. 1903, *Transactions*, A. S. M. E., Vol. 45 (1923), p. 343.

[16] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), Fig. 4, p. 773.

profile of the velocity. By taking sufficient readings of velocity at various positions vertically, the average velocity of the entire passage is determined. One horizontal velocity profile is taken by simultaneous readings of the meters

This method of water measurement is believed to result in too small a value of discharge, for reasons that have been considered carefully. Ratings in still water with the meter turned at an angle to the direction of travel show invariably that the spoke-type axial-flow meters, such as those recommended for turbine tests, register less than the component of the velocity in the direction of the meter axis, which is obviously the velocity of travel in the water, times the cosine of the angle at which the meter is turned. Not only is this a demonstrated fact, but a graphical analysis of the angle of attack of the water on the blade, when in two opposite positions of its rotation, show that, for reasonable angles of inclination, the degree of under-registration agrees with that which would be expected.

Few intakes of modern units have walls that are parallel at the measuring section, from which fact there is necessarily some obliquity of flow. Consequently, the meter is expected to register, not the true velocity, but the component perpendicular to the measuring section, which from the still-water ratings, and from theoretical analysis, cannot be done accurately by any known meter. Some attempts have been made abroad to turn the meter to agree with the theoretical streamline, and then to correct for the angle that this makes with the metering section.[17]

The situation becomes even more complicated when it is considered that turbulence may cause the meter to be buffeted by a multiplicity of impulses coming possibly from widely varying angles and directions; and no matter how much falsework may be constructed to straighten the flow, there are always eddies and cross-currents.

Recognizing the fact that some meters under-register the flow, a valuable suggestion was made by W. Monroe White, M. Am. Soc. C. E., that a meter known to under-register might be used in conjunction with one that is known to over-register; and, by combining the results, the true flow might be determined. The suggestion proved impractical for the simple reason that there is no meter which, when rigidly supported, will over-register consistently the same amount for a given angularity regardless of the direction from which the flow comes. Nevertheless it was the basis for an important technical advance in the measurement of water.

Failing to find two meters that had opposite characteristics, the next step which followed very logically was to find two under-registering meters that had differing characteristics. J. M. Mousson demonstrated[18] that the apparent failure of certain current meters to agree with the volumetric standard could be accounted for easily by their differing oblique flow characteristics in spite of the fact that the channel upstream from the meter had a uniform cross section for a long distance. Later he demonstrated[19] that the quantity of

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[17] "Eine neue Anwendung des Flugelmessverfahrens bei den Abnahmeversuchen im Limmat-Kraftwerk Wettingen," by C. F. Streiff and H. Gerber, Schweiz-Bauzeitung, Vol. 103, No. 3, pp. 36–39.

[18] Discussion of paper "Research Institute for Hydraulic Engineering and Water Power," by Hunter Rouse, Assoc. M. Am. Soc. C. E., *Transactions*, A. S. M. E., Vol. 55, No. 10, August, 1933, pp. 35–39.

[19] *Transactions* ,A. S. M. E., February, 1936, Vol. 58, p. 141.

under-registration of spoke-vane meters of the same family was directly related to the pitch, and again graphical analysis of the angle of attack explained within reasonable limits the results that were actually observed in the rating tank.

The outstanding fact that was revealed, both experimentally and on paper, was that, regardless of the angle at which a spoke-vane meter is inclined to the actual movement of the water relative to it, the extent of under-registration of one meter in comparison with that of the other is in direct proportion to the pitch.[19] Consequently, assuming that both meters are exposed to exactly the same conditions of flow, the difference in registration becomes a measure of the extent to which either of them under-registers the true flow. It is interesting to note (again within reasonable limits) that the angle at which the water strikes the meter, whether uniform or constantly varying in amount and direction, drops completely "out of the picture" since by the law of averages both meters will be subjected to the same average conditions in the course of a test.

To be sure, this method falls short of Mr. White's ideal for the reason that, unfortunately, it is necessary to extrapolate outside of two readings rather than to interpolate between them; but at least it makes use of means at hand instead of abandoning the effort because something is needed that does not exist. This method was first used[20] in making turbine efficiency tests on the 42,500-hp Kaplan turbines at Safe Harbor.

Since this method of water measurement was new, it was deemed advisable to compare its accuracy with some better-established method. After numerous experiments at Niagara Falls, N. Y., N. R. Gibson, M. Am. Soc. C. E., found that he would have confidence in his ability to measure, satisfactorily, the discharge of the Safe Harbor units with the length of test section available. Consequently, two units were equipped with the necessary piezometer taps, and the tests were conducted. The agreement between the Gibson tests and the two-type, current-meter tests was excellent, being less than 1%. The maximum discharge of these units is about 10,000 cu ft per sec.

In spite of the excellent agreement between these two comparative tests, there is still doubt in the minds of some hydraulic engineers regarding the fundamental principle of the two-type, current-meter method. These doubts arise partly from the assumption that turbulence in flowing water affects the meter the same as angularity in the still-water rating. To date (1939) there has been neither conclusive proof that this assumption is correct, nor proof to the contrary.

An acceptance test by the current-meter method requires rather expensive equipment, most of which cannot be adapted, economically, to tests on other developments. For this reason, it may be anticipated that the cost of a current-meter test will be considerably greater than a test by the Gibson or Allen method. Consequently, the use of current meters will be limited to low-head developments with extremely short intake passages where the other two methods cannot be applied.

[20] "Water Gauging for Low Head Units of High Capacity," by J. M. Mousson, *Transactions*, A. S. M. E., August, 1933.

Messrs. Allen and Gibson have found from experience that years of study and practice are necessary to train men to conduct tests successfully by their methods. The two-type, current-meter method is no less intricate than the other two methods and requires every bit as much training before an engineer can be considered competent to conduct tests. To a large extent the success of the Allen and Gibson methods has depended upon the close control which they have kept over the application of their methods, permitting only competent assistants to conduct tests that they were unable to supervise personally. Since the two-type, current-meter method is not a patented process, no such strict control is possible, although highly desirable.

Conclusions

The engineering profession is quite likely to have a self-satisfied feeling regarding the state of the art of turbine design similar to the feeling which was prevalent about 1914 when turbine efficiencies of more than 90% were attained. However, a review of the progress during the intervening years will show the fallacy of that feeling, and a study of the possibilities for improvement on present designs should spur engineers to renewed efforts.

As in the past, the laboratory is the logical place to study means of improvement, although theoretical studies should not be neglected. Since the margin of possible gain is relatively small, it is of considerable importance to take every precaution to eliminate all sources of error in the laboratory. In the past there has been the tendency to make assumptions in order to reduce the cost of tests, finding later that the accuracy of the tests was questionable because of the unjustified assumptions. It is equally important to make tests on the prototype so that there will be assurance that the expected performances are actually attained.

It would be of great assistance to the profession if each manufacturer would make a study of step-up with an attempt to determine whether the exponent for the ratio of turbine diameters in the step-up formula should be 0.25, as used by Moody, or 0.20 as indicated from the theoretical analysis by Professor Pardoe.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

EFFECTS OF RIFLING ON FOUR-INCH PIPE TRANSPORTING SOLIDS

By G. W. Howard,[1] Jun. Am. Soc. C. E.

Synopsis

Tests to determine the effects of rifling installed in 4-in. pipe upon the transportation of mixtures of water and material are described in this paper. Results were compared with those from 2-in. pipe in an effort to discover any similarity between the transportation characteristics of each pipe such that the principle of these smaller pipes could be applied to larger pipe. Tests for the development of the optimum design of rifling for pipes in which sand was the transported material constituted the major part of the testing program. The rifling was tested later with silt, clay, and pea gravel used as the transported materials.

An attempt is made to show that the results obtained through the study of 2-in. and 4-in. pipe can be extended to include pipes having a larger diameter; and further, that, under certain conditions, dredging technique of the present day can be improved considerably by the proper use of rifling in the discharge line.

Introduction

Material transported in pipe lines has been found to concentrate in the lower part of the pipe.[2] Knowledge of this phenomenon caused the Memphis (Tenn.) Engineer District, in 1913, to experiment with a mixer to provide uniform distribution of the material over the cross section of the pipe (see Fig. 1). The mixer that was tested consisted of a diametrically warped plate, with its upstream edge horizontal, fastened in the pipe. The pitch of this mixer was 10 pipe diameters and its length was 5 pipe diameters. Fastening this plate in the pipe effected a division of the contents into two parts—an upper and a lower half. In passing through the mixer, the two halves of the

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **March 15, 1940.**

[1] Associate Engr., Office, Chief of Engrs. U. S. Army, Washington, D. C.

[2] *Transactions*, Am. Soc. C. E., Vols. LVII (1906) and 104 (1939), pp. 403 and 1334, respectively. See also *Professional Memoirs*, Corps of Engineers, U. S. Army, March–April, 1915.

contents reversed their positions: The upper part became the lower part, and the lower part became the upper part. A horizontal plate, having a length of 3.8 diameters, was added to prevent the material from being carried down to the bottom of the pipe as it emerged with a spiral motion; the purpose of this plate, then, was to deliver the material with its motion parallel to the axis of the pipe, but with its position changed from the lower half to the upper half.

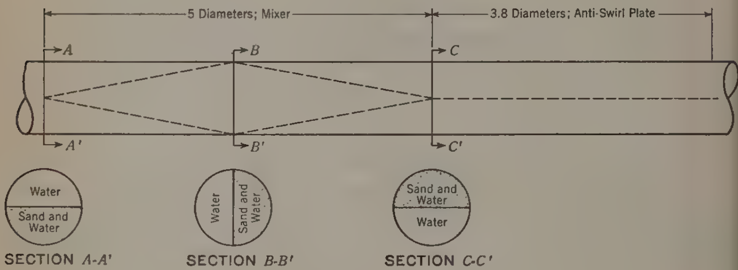

Fig. 1

Tests on this mixer showed that it was successful in locations where pure sand was encountered—that is, in material where no tree stumps were embedded. The reduction of half the effective area of the pipe for passing stumps caused considerable difficulty in blocking the line. As a result of this limitation, tests on the mixer were abandoned. However, investigation on the warped plate mixer led to a comprehensive series of experiments at the U. S. Waterways Experiment Station, during the period 1935–1937, to develop a mixer for installation in pipe lines.

Program of Experiments

Two separate series of tests were performed. The first series included tests on 4-in. pipe to determine the optimum design of rifling for use in a pipe line transporting sand. Results obtained from this design were checked with those from tests on 2-in. pipe. The second series included tests to determine the characteristics of the optimum rifling when materials other than sand were transported. For these tests the materials used were (*a*) silt, (*b*) clay balls, and (*c*) pea gravel.

Set-Up for Experiments

The apparatus[3] used for these investigations is shown diagrammatically in Fig. 2. Briefly, the apparatus consisted of a circulating system wherein water, or water mixed with material, was pumped from a sump to a variable head tank, from which it was discharged through the test pipe line back into the sump. Manometers were placed above and below the section of pipe being

[3] For a complete description, see *Transactions*, Vol. 104 (1939), p. 1334.

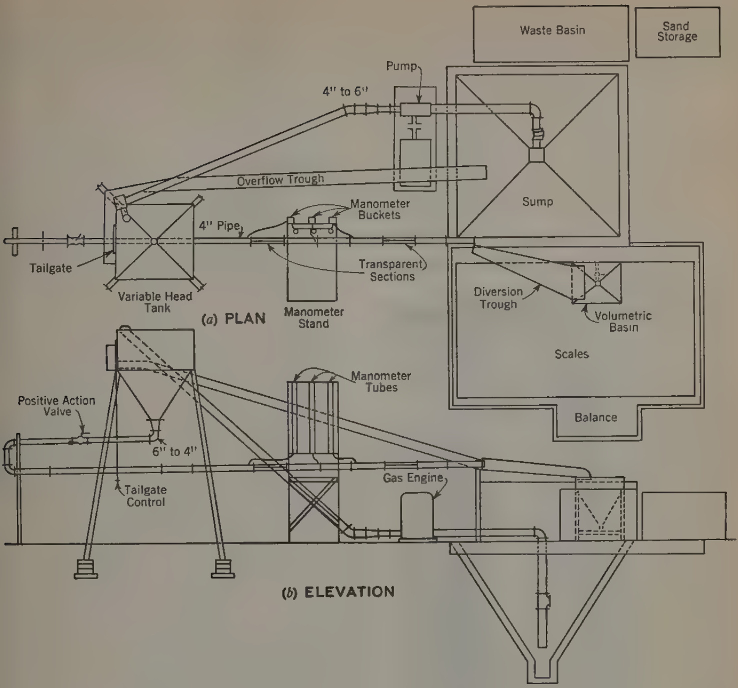

Fig. 2.—Apparatus Used on Tests for Sand and Gravel

TABLE 1.—Specifications for Rifling

(a) First Series

Type	Number of rifles	Dimensions of Rifles, in Inches: Width	Dimensions of Rifles, in Inches: Height	Dimensions of Rifles, in Inches: Pitch	Spiral, in degrees	Total length, in pipe diameters
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	1	1/8	1/2	40	180	5
2	1	1/16	1/2	40	180	5
3	1	3/16	1/2	40	180	5
4	1	1/8	1/4	40	180	5
5	1	1/8	3/4	40	180	5
6*	1	...†	...†	40	180	...†
7	1	1/8	1 1/2	20	54	1 1/2
8	3	1/16	3/4	40	360‡	10
9	3	1/16	3/4	20	360‡	5
10	3	1/16	3/4	40	180	5
11	3	1/16	3/4	20	180	2 1/2
12	3	1/16	1/2	40	360‡	10
13	3	1/16	3/4	40	54	1 1/2
14	3	1/16	1/4	120	....§	43 1/3
15	3	1/16	3/8	80	... §	{43 1/3, 86 1/4}

(b) Second Series

Type	Dimensions of Rifles, in Inches: Height	Dimensions of Rifles, in Inches: Pitch	Total length, in pipe diameters
(1)	(4)	(5)	(7)
16	1/2	40	2 1/4
17	3/4	40	2 1/4
18	3/4	30	2 1/4
19	1/2	40	6 1/2
20	1/2	40	4 1/2
20-A	1/2	40	5 1/2
21	1/2	60	6 1/2
23	1/2	60	4 1/2
24	1/2	40	8

* Design in original field tests, in 1913.
† Diametrical plate; 15 in. of straight plate at end of warped section.
‡ Rifles spaced at 120° around the pipe.
§ Continuous.

tested. Provision was made for the diversion of the discharge from the pipe line into the volumetric basin supported on scales. By analysis of the mixture thus diverted, the percentage of solids which was being moved was determined.

Test Sections

Specifications.—The scope of the investigation included changes in number, width, height, pitch, spiral degrees, and length of rifling. Specifications for each of the types of rifling tested are shown in Table 1.

In Table 1(*b*), for all types except 20-A, the total length of the rifling was divided into two parts and inserted in each end, leaving a space of 3 in. between rifle ends in the adjoining pipe lengths. Type 20-A differed in that all the rifling was inserted in the upper end of the 6-ft 8-in. pipe length.

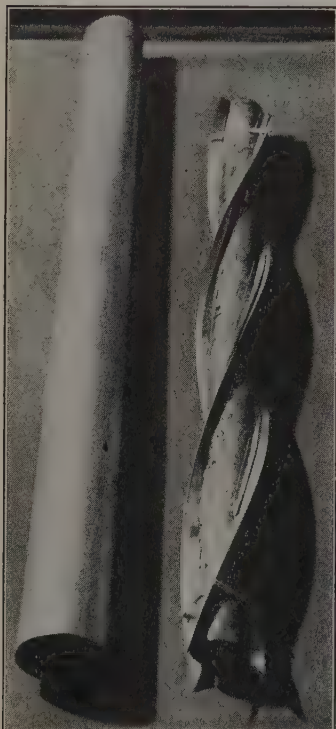

Fig. 3

All the mixers in Table 1(*b*) had the common specification of three rifles spaced at 120° around the section and a $\frac{1}{16}$-in. width of rifle. The ratio of rifling to plain pipe in types 19 and 24 was 1 in 3. Over-all pipe lengths of 24 diameters (type 19) and 20 diameters (type 24) were for approximate simulation of commercial lengths of 20-in. and 32-in. discharge pipes used on dredges.

Type 6, Table 1, is the same design as that used in 1913 on the dredge "Iota." It was composed of a diametrical plate with a 40-in. pitch, warped through 180°, and with a straight plate on the end for the next 15 in. In types 8, 9, 12, and 13, the rifles were spaced at 120° around the pipe. The single rifle in type 7, Table 1, was made like a piston so that it could be removed from the line or set at any height. In types 10 and 11, the rifles were spaced at 60° around the pipe, the center rifle starting on the bottom of the pipe.

The rifles were spaced at 120° around the pipe in the case of types 12 and 13, Table 1; and in types 14 and 15, the height of rifles was $\frac{1}{2}$ in. for the first 10 diameters and $\frac{1}{4}$ and $\frac{3}{8}$ in., respectively, for the remaining length. Type 15 was tested twice, first at $43\frac{1}{3}$ diameters and next at $86\frac{1}{4}$ diameters.

Design and Construction.—In order to obtain a smooth finish inside the pipe after the installation of rifling, it was necessary that each rifle fit flush along

the pipe, prior to soldering. The pipe itself was made from 20-gage sheet metal and the rifles were of 16-gage metal. Although it was possible to bend these strips of metal and obtain an approximate fit, the absence of an exact fit would permit a small amount of solder to enter beneath the strip and cause enough bulge to change the height. Therefore, each strip was designed as a segment of a cone with the distances along the upper and lower edges of the rifle as diameters. The slant height of this segment was the height of the rifle, and the total slant height of the cone was the radius used to describe the arc along which to cut the metal.

A metal core was used to hold the rifles in place during the process of soldering inside the pipe. Grooves on the outside of this core maintained the correct pitch and, after fastening one end of each rifle, the core guiding the strips into place was twisted through the section. Fig. 3 shows the arrangement used for installing the rifles immediately before placing the core inside the pipe.

Procedure of Experimentation

The attempt to discover the optimum rifling led to the conclusion that tests should be conducted which involved the determination of the following: (*a*) The characteristics (head loss versus velocity) of a pipe, without rifling, carrying a sand and water mixture; (*b*) the characteristics of different designs of rifling when identical sand and water mixtures were transported; (*c*) the effect of changes in the size of the pipe upon its transportation characteristics; and (*d*) the characteristics of the optimum rifling when transporting mixtures of water and silt, water and clay balls, and water and pea gravel.

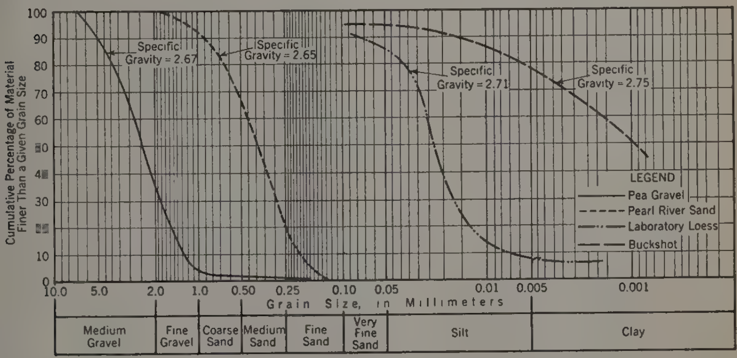

Fig. 4.—Mechanical Analyses of Materials Tested

Determination of concentration, velocity, and pressure were the primary features in test operation. Solid concentration was determined from the weight of a known volume of the mixture, and pressures were measured with manometers connected by tubes to piezometers (the manometers were installed in the same location for all test sections, providing direct comparisons for the different types of rifling).

TEST RESULTS

Sand Tests, 4-In. Pipe.—The material used in the investigation was known locally as Pearl River sand. A mechanical analysis of the material is shown in Fig. 4. Investigations in 4-in. plain pipe showed that a velocity range of from 6.0 to 8.5 ft per sec covered all types of sand transportation in pipe lines.[3] Accordingly, this range was used for tests on mixers.

The initial investigations included tests from type 1 through type 12 (see Table 1(*a*)). In these tests the design of rifles was of primary importance—that is, the width, height, pitch, and spiral. Of the designs that were tested,

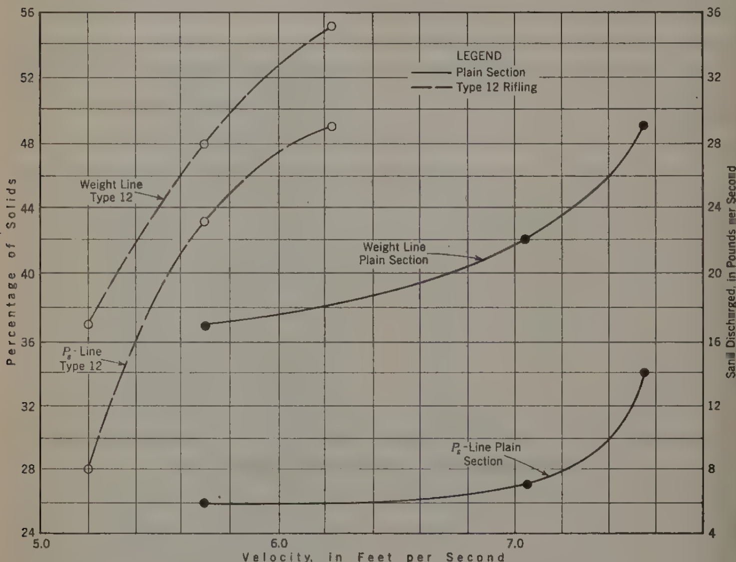

FIG. 5.—RELATION BETWEEN "BLOCKING" POINTS FOR A PLAIN SECTION AND FOR TYPE 12 RIFLING

type 12 was considered the most satisfactory. The relation of the "blocking points," or points of clogging, for this rifling to those for plain pipe is shown in Fig. 5. The head loss determination for type 12 compared to plain pipe is shown in Fig. 6.

Since this mixer proved satisfactory, the question of spacing in the line was the next to be considered. Numerous spacings were tested in an effort to determine which location would be satisfactory. The results of this investigation, for the optimum ratio of rifling to over-all pipe length, are presented graphically in Fig. 7. From these results it was determined that the length of rifling should be one-third the length of the over-all pipe section. Note that all velocity curves shown on this sheet are for center-line velocities. The dimensions of type 12 rifling are given in Table 1(*a*).

Investigation was next made of pipe lengths of $19\frac{1}{2}$ and 24 diameters. (The length of commercial pipe has diameters of 32 in. and 20 in., respectively.) From this investigation a rifling was designed which was known as type 24 and had specifications as follows: 3 rifles spaced at 120° around the section; rifling $\frac{1}{8}$ diameter in height, and 10 diameters in pitch; and length of pipe having rifling one third the length of the over-all pipe section (pipe sections tested were $19\frac{1}{2}$ and 24 diameters long); this length was divided in two and inserted in each end of the pipe. The curves for head loss for this rifling and plain pipe are shown in Fig. 8(*a*).

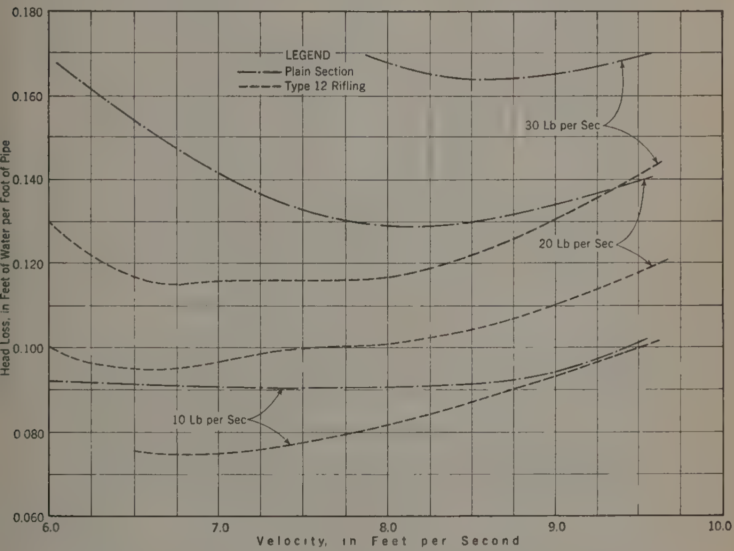

FIG. 6.—LOSS OF HEAD PER FOOT OF PIPE FOR A PLAIN SECTION AND FOR TYPE 12 RIFLING

Sand Tests, 2-In. Pipe.—The results from the 4-in. pipe were then compared with those obtained from a smaller (2-in.) pipe. In order to insure a basis for comparison, the following conditions were observed during the latter tests: (*a*) The rifling used in the 2-in. pipe was similar geometrically to that used during tests on type 24 rifling in 4-in. pipe; and (*b*) the sand tested during the studies on 2-in. pipe was identical with that used for the 4-in. pipe tests. No effort was made to obtain similarity of flow in the pipe; and the material moved in the 2-in. pipe was identical with, and not similar to, that carried in the 4-in. pipe. Transportation characteristics of the pipe lines were the major consideration of the study. Hence, no precise laws of similitude could be applied in analyzing the results. On the contrary, only general comparisons could be made.

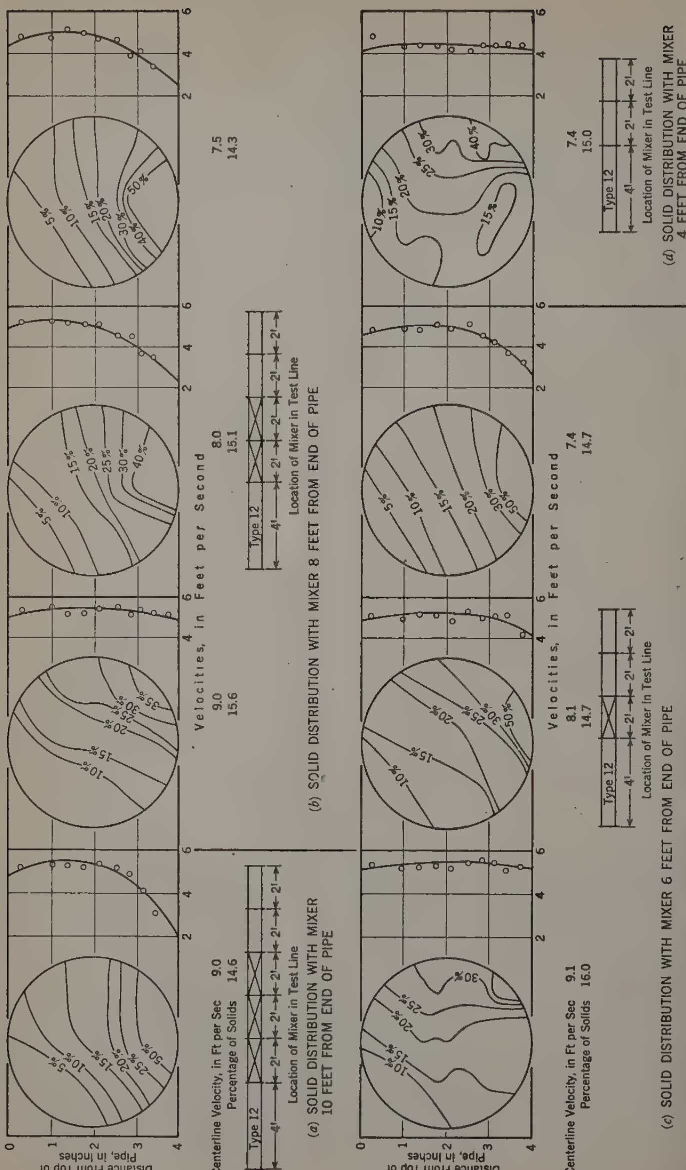

FIG. 7.—SPACING TEST, TYPE 12; OBSERVATIONS OF SOLID DISTRIBUTION AT END OF 4-IN. PIPE

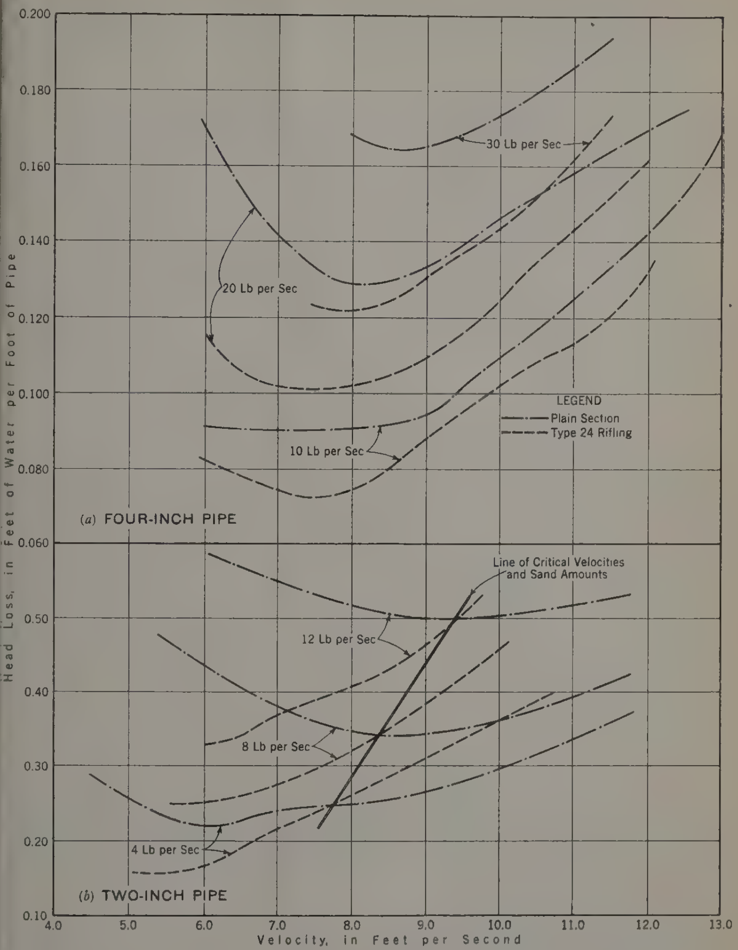

FIG. 8.—LOSS OF HEAD PER FOOT OF PIPE IN A PLAIN SECTION AND WITH TYPE 24 RIFLING

Fig. 8(*b*) presents, graphically, the results of the tests on 2-in. pipe. It is to be noted that the curves for the rifled and plain pipe intersect. At the lower velocities the rifled pipe has a superior efficiency, whereas the reverse is true of the higher velocities. The point at which the efficiencies are identical (that is, the point of intersection of the curves in Fig. 8(*b*)) is called the point of "critical velocity."

Comment.—Observation of the movement of the sand through plain pipe (the observation being made through a transparent section of the pipe—see Fig. 2) indicates the following: (*a*) Below a velocity of about 4.5 ft per sec, the sand settled on the bottom of the pipe; (*b*) above a velocity of about 7 ft per sec, the sand was moved freely, there being enough turbulence to keep all particles in motion along the bottom of the pipe; and (*c*) between the velocities of 4.5 and 7 ft per sec, the sand moved along the bottom in jerks.

With reference to the results of the tests of type 24 rifling on 4-in. pipe (Fig. 8(*a*)) it is noted that, throughout the range of velocities tested, the rifled pipe showed a superior efficiency as compared to that of plain pipe; that is, for that particular test, no point of critical velocity was reached. Observation of the type of movement in the 4-in. pipe indicated that material settled at velocities of less than about 6 ft per sec, and that the material moved freely at velocities of more than about 7.5 ft per sec. Hence, so far as type of movement is concerned, the velocity range is 6.0 to 8.5 ft per sec in the 4-in. pipe.

Since no laws of similitude are applicable in comparing the data from the 2-in. pipe tests with those from the 4-in. pipe tests, it appears that the best that can be done is to observe the general behavior of each over that range of velocities where similar types of movement occurred. Such general comparison indicates, in each case, a superior efficiency for rifled pipe up to the velocity at which material begins to move freely in the pipe. At higher velocities, the rifled 2-in. pipe loses its superiority and becomes less efficient than plain pipe. In the case of the larger pipe, however, the superior efficiency of the rifled section is maintained for velocities considerably in excess of the one at which material begins to move freely.

Tests to Determine the Effects of Rifling on Pipe Carrying Material Other than Sand

After determining that rifling would produce beneficial effects when a sand of large-grain diameter was transported, the problem was resolved into a determination of the effects when other materials were being transported. Accordingly, tests were made using silt, clay, and pea gravel as the transported materials.

Silt.—The material used in this study was a loess obtained on the laboratory grounds, the mechanical analysis of this material being shown in Fig. 4. The silt was transported through the pipe in suspension, the completeness and evenness of which demonstrated that the distribution of solids was practically constant over the cross section. This phenomenon of transport in suspension in the case of silt is radically different from that in the case of sand; the latter is carried mostly in the lower part of the pipe line. The curves of Fig. 9(*a*)

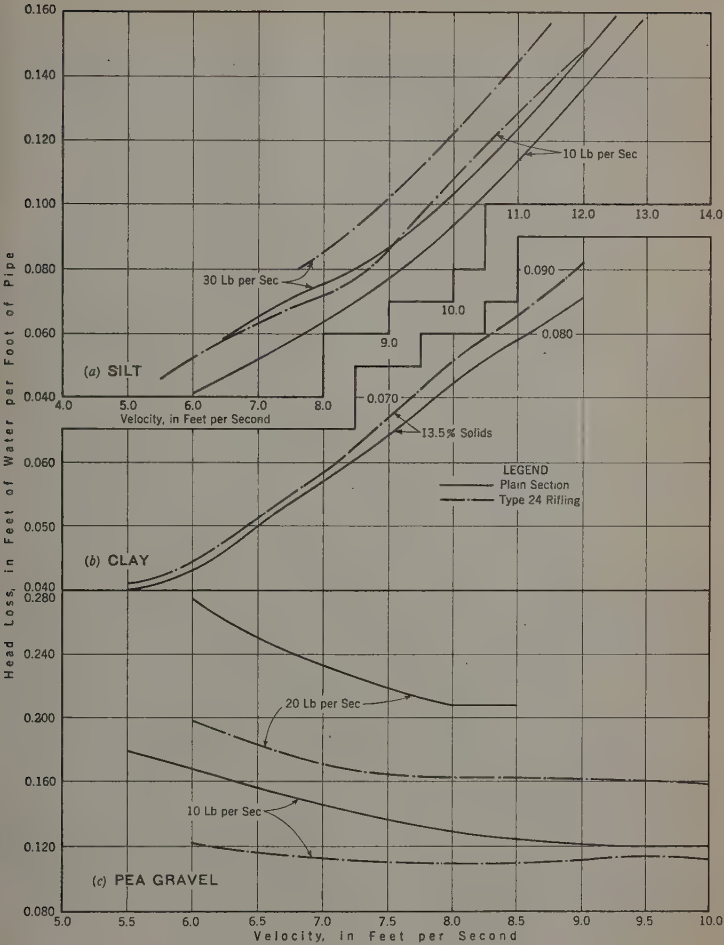

FIG. 9.—LOSS OF HEAD PER FOOT OF 4-IN. PIPE TRANSPORTING VARIOUS MATERIALS

are presented to show comparable characteristics of plain and rifled pipe carrying silt at various velocities and in identical quantities. They indicate superior efficiency for the plain pipe.

Clay Balls.—The material used in this study was a clay, known locally as "buckshot." Results of the mechanical analysis of the material are shown in Fig. 4. For use in the tests the material was molded in the form of balls, having diameters as great as 1.5 in. The intention was to simulate the clay lumps which are sometimes encountered by dredges in the field. All the tests with clay were conducted with a mixture containing 13.5% solids. The attempt was made to ascertain whether or not rifling would have adverse effects on the passage of the clay balls through the pipe. The efficiencies of rifled and plain pipes in the transport of this material were determined also.

Direct observation of the effect of rifling on the clay balls was very difficult because of the turbidity of the water. After each test, however, the rifles were inspected. As far as could be determined, there was no tendency for the balls to be caught on the ends of the rifling. The relative effects of rifling on the efficiency of a pipe line when carrying such clay balls are shown in Fig. 9(*b*). It is to be noted that the plain pipe is more efficient than the rifled one.

Pea Gravel.—The material used in these tests is known locally as pea gravel (see Fig. 4). In general, the material is described as a gravel with no particle greater than $\frac{1}{4}$ in. in diameter. The method by which the gravel was transported along the pipe was determined by observation through the transparent section. It was seen that the gravel settled quickly and moved generally, without jerking along the bottom of the pipe.

The curves of Fig. 9(*c*) are presented to show the effects of rifling on the efficiency of a pipe line carrying pea gravel. It is to be noted that throughout the range of velocities tested the rifled pipe line shows an efficiency higher than that of the plain pipe. It is to be noted also that the curves are similar to those already determined for pipe lines carrying sand (see Fig. 8(*a*)).

Comment.—The experiments indicate that rifling in a pipe line increases the efficiency of the pipe in transporting materials that are large and heavy enough to settle and travel along the bottom of the pipe (sand and gravel). This statement must be qualified as follows: As velocities and turbulence increase, any given material tends more and more to move in suspension rather than along the bottom. As this tendency increases, the relative efficiency of the rifling in the pipe line decreases. Finally, a point is reached where the resistance to flow offered by the rifling more than offsets the benefits derived from the mixing effects of the rifling. Beyond this point the rifled pipe is less efficient than the plain. This phenomenon is best illustrated by the data from the tests on 2-in. pipe carrying sand (Fig. 8(*b*)).

Use of Rifling on Hydraulic Dredges

In order to use rifling effectively on dredges on the Mississippi River, it is believed that the only satisfactory installation would be on channel-maintenance dredges which work in coarse sand and small gravel the greater part of their operating time. When material favorable for the use of rifles is being

dredged, rifling a pipe line can be considered equivalent to installing a booster pump in the line. Thus, for long pipe lines or low-powered dredges, it would be possible to increase production without making mechanical installations on the dredge itself.

It is desired particularly to emphasize the point that the justification for rifling a discharge line is dependent upon the class of material encountered and the power of the pumping unit. There is a different problem on each dredge, therefore, and a careful estimate of the situation should be made in respect to the power of the dredge and the class of material to be encountered before it is decided to make any installations of rifling.

Conclusions

From consideration of the investigation described herein, the following conclusions are believed justified:

(1) Rifling will increase the efficiency of the line when the following materials are transported: (*a*) Coarse sand, and (*b*) gravel.

(2) Rifling will reduce the efficiency of the line when the following materials are transported: (*a*) Silt, and (*b*) clay.

For general use in considering the advisability of the use of rifling, the following is offered as a criterion:

> Rifling in the discharge line of a dredge will increase the efficiency of the line in cases where the material being dredged through a plain pipe would settle along the bottom in appreciable quantities.

Acknowledgments

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PAPERS

TRANSIENT FLOOD PEAKS

By Henry B. Lynch,[1] M. Am. Soc. C. E.

Synopsis

Floods of the so-called "cloudburst" type yield momentary runoff peaks entirely out of proportion to the rate of rainfall. They are caused by an abrupt increase in rainfall and runoff. Their magnitude is controlled by many factors, of which probably the most important are the rate of increase and the intensity of the rainfall. One of the most noteworthy examples of this type of flood was the flood near Los Angeles, Calif., on the morning of January 1, 1934.

The more detailed study from which this paper was prepared has been placed on file for reference at Engineering Societies Library.[2]

General

An abrupt increase of rainfall, under some conditions, produces a transient runoff peak from a small area exceeding by many fold the flow which could be sustained by the rainfall upon the area. In computing flood flows it is usual to consider that the peak discharge is at some rate less than the rainfall on the area, and that runoff never exceeds rainfall except in the case of a temporary damming of the channel. This is true of the runoff from a rain of uniform intensity, and it is true also of the sustained runoff in all cases. It is sometimes entirely untrue of the momentary maximum runoff when the rainfall is increasing suddenly. With a runoff that is growing greater there is a tendency for the deeper, faster moving water from upstream to overtake the water in front of it. When the increase is abrupt there may accumulate, at the junction of the fast and slow moving waters, a steep-fronted surge popularly termed a "wall of water" many fold larger than the sustained runoff behind it. The same effect occurs, under conditions favoring it, at the beginning of an exceedingly abrupt and heavy rain. In large areas the runoff seldom increases rapidly enough for these surges to occur.

Under usual conditions, where rain falls steadily over the entire area of the canyon, the depth and quantity of runoff increases continuously from the head of the canyon to its mouth. However, when an abrupt, heavy rain begins

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by March 15, 1940.

[1] Cons. Engr., Los Angeles, Calif.

[2] 33 West 39th Street, New York, N. Y.

near the head of the canyon, under circumstances such as to favor a rapid runoff, a reversal of normal runoff conditions may exist temporarily in the upper part of the canyon. Under such conditions, the water upstream may be deeper than the water in front of it, for a short time, and runoff may take some such form as is shown in Fig. 1. Abrupt rainfall, concentrated near the head of the canyon, is not uncommon. It is the most favorable condition for the formation of the surge. Under conditions favoring it, the surge builds up to a size, and to a destructive power, which seem entirely out of proportion to the rate of rainfall.

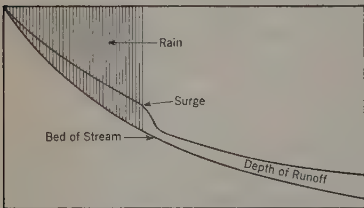

Fig. 1.—Abnormal Runoff from Rain at Head of Canyon

An increase of rainfall near the head of the canyon is not the only cause that can produce this reversal of normal runoff conditions. The topography of the canyons in Los Angeles County (see Figs. 2 and 3) which emitted surges on January 1, 1934, is such that an abrupt increase of rainfall covering the entire canyon might cause a surge almost as violent as would be the case where the rainfall increase occurred only near the head of the canyon.

There have been many floods of this type in all mountainous regions of the United States. In the ten years, 1929 to 1939, six of them can be definitely identified as having occurred in Southern California. In all but one of these

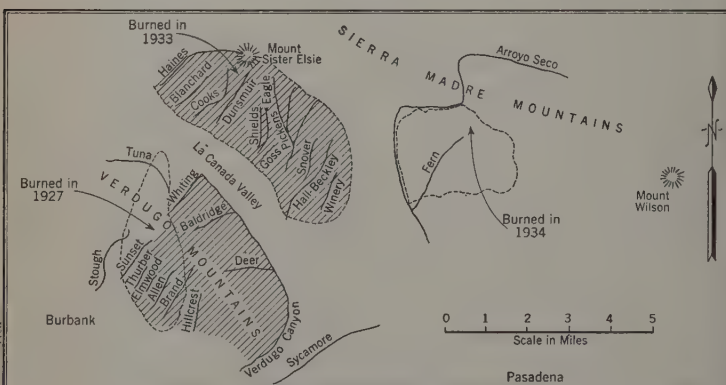

Fig. 2.—Key Map of a Part of Los Angeles County

cases, the floods were confined to a small area, with no loss of life, and comparatively small property damage.

All the canyons from which these surges emerged were upstream from thickly settled suburbs. In several cases automatic rain gages were located

within the canyons which generated the surges, and in each of the other cases automatic gages were in operation nearby.

Flood of January 1, 1934

The most destructive and extreme examples occurred in the mountains in Los Angeles County, California, on January 1, 1934. During the entire day and evening of December 31 a heavy rain had fallen, coming from the south, and thoroughly soaking the ground. Before evening, the thin soil covering the

Fig. 3

steep mountains had attained a high degree of saturation, and runoffs were large. Just before midnight the rain suddenly increased for a short time from a rate of about 0.5 in. to about 1.4 in. per hr across a five-mile front which included parts of the Sierra Madre and of the Verdugo mountains to the south. The same thing occurred, but to a lesser extent, in other parts of the county.

The response of the runoff was prompt and violent. Within ten minutes before and after midnight surges of water, heavily laden with debris, emerged from each of the steeper canyons. These surges were many fold larger than

could have been sustained by the rainfall upon the area. None of the canyons drains an area as great as 2 sq miles, and only three have areas greater than 1 mile. The surges emerged first from the canyons to the west. In Sierra Madre, the surge from Hall-Beckley Canyon seems to have been about twenty minutes later than that from Haines Canyon. The magnitude and violence of these surges are difficult to describe or picture adequately. Some of the canyons in the Sierra Madre showed flow sections which would seem incredible if they were not amply established by numerous photographs, measurements, and observations. In the period since this flood most of the high water marks have disappeared, but enough of them were remaining in 1939 to show how excessive these sections were when compared to the sections which would be usually expected. The rainfall intensity at the time of the formation of the surges was far below rates which have been recorded many times, both in Los Angeles County and elsewhere. Except for the moderate rate of rainfall, all of the conditions in the Sierra Madre favored the formation of the surges to a high degree. These mountains had been burned over a few weeks before the flood and were completely denuded of vegetation in most of their area within the zone of excess runoff.

The term "zone of excess runoff," as used herein, refers to the area which yielded runoff peaks greater than could be sustained by the rain falling upon the area (see areas in Fig. 2). Much of the area referred to herein as the "normal zone" showed rainfalls of similar amount and characteristics, but one or more of the factors necessary to the formation of the surges at such moderate rainfall rates were lacking.

These surges were confined to the zone of greatly increased rainfall on the south slope of the Sierra Madre and in the eastern portion of the Verdugo Mountains. They covered a roughly triangular area 5 miles on a side. The western part of the Verdugo Mountains gave normal runoffs, as did also Sycamore Canyon, about 1 mile east of these mountains. The surges showed a definite gradation from the great heights in the recently burned areas to the normal runoffs in the unburned areas outside of the zone of excess runoff. The greatest surges were those in the then recently burned area. Then followed in order, with some overlapping, the surges in the 1927 burn in the Verdugo Mountains, within the zone of excess runoff; the surges in the unburned areas in the zone of excess runoff; and the runoffs both in the burned and unburned areas outside of the zone of excess runoff.

Flow sections were measured in thirty-one places, and included twenty-two canyons or branches in the zone of excess runoff. In Table 1 the canyons have been segregated into the foregoing classifications. The unit used is the cross section of the flow for each square mile of catchment area. No attempt has been made to compute the peak flows.

In the zone of normal runoff, velocities and discharges have been computed by the Manning formula for determining the flow in open channels:

$$V = \frac{1.486}{n} R^{0.67} S^{0.50} \qquad (1)$$

TABLE 1.—Hydraulic Properties of Sections Involved in the Flood of January 1, 1934

Sections	Drainage area (see Figs. 2 and 3)	Without Check Dams						With Check Dams			Cross-sectional area per square mile of catchment
		Area of cross section A, in square feet	Feet: Wetted perimeter, P	Feet: Hydraulic radius, R	Coefficient of roughness, n†	Hydraulic slope, S	Catchment area, in square miles	Area of cross section A, in square feet	Feet: Wetted perimeter, P	Feet: Hydraulic radius, R	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(2)	(3)	(4)	(10)
	(a) Areas Burned in 1933										
A	Winery	26	29	0.9	0.030	....	...	....	...	...	....
		130	29	4.5	0.040	0.080	0.14	....	...	...	1,114
B	Hall-Beckley	813	101	8.0	0.040	0.069	0.72	680	99	6.9	944
C	Pickens:										
C-1	Above Mullaly	456	61	7.5	0.025	0.100	1.28	....‡	...‡	...‡	....
C-2	Mullaly Fork	536	59	9.1	0.035	0.077	0.35	493	57	8.7	1,408
C-3	Above Gould's Castle	711	81	8.8	0.050	0.087	1.78	621	73	8.5	349
C-4	Gould's Castle	521	71	7.3	0.050	0.090	1.78	406	66	6.1	222
C-5	Mountain Street	693	80	8.7	0.035	0.072	1.80	590	77	7.7	328
C-6	Cross Street*	1,231	298	4.1	0.045	0.067	1.81	1,166	292	4.0	644
D	Goss	64	24	2.7	0.035	0.074	0.28	....	...	...	229
E	Eagle:										
E-1	East Fork	66	23	2.9	0.055	0.133	0.06	26	18	1.4	433
E-2	West Fork	154	33	4.7	0.055	0.164	0.16	....	...	...	962
F	Shields	296	46	6.4	0.050	0.121	0.24	212	41	5.2	883
G	Dunsmuir	1.134	214	5.3	0.050	0.104	0.86	1,016	210	4.8	1,181
H	Cooks	625	77	8.1	0.035	0.088	0.65	473	74	6.4	728
I	Blanchard	305	72	4.2	0.035	0.061	0.45	....	...	...	677
J	Haines	299	49	6.1	0.040	0.122	1.20	....	...	...	239
	Average	....	...	...	....	....	...	....	...	...	689
	(b) Burned in 1927; Zones of Excess Runoff										
K	Brand:										
K-1	Total flow	80	22	3.6	0.014	....	...	....	...	...	....
		153	90	1.7	0.030	0.060	1.06	....	...	...	220
K-2	East Fork	180	38	4.7	0.055	0.089	0.43	....	...	...	418
L	Allen:										
L-1	Total flow	103	79	1.3	0.030	0.037	0.34	....	...	...	303
L-2	West Fork	83	27	3.1	0.035	0.059	0.22	....	...	...	377
M	Thurber	10	9	1.1	0.030	0.084	0.03	....	...	...	333
N	Elmwood	15	25	0.6	0.015	0.049	0.33	....	...	...	45
	Average	....	...	...	....	....	...	....	...	...	283
	(c) Unburned Canyons; Zones of Excess Runoff										
O	Whiting	57	20	2.8	0.040	....	...	....	...	...	....
		19	11	1.7	0.055	0.034	0.84	....	...	...	90
P	Baldridge	116	38	3.0	0.035	0.054	0.73	....	...	...	159
Q	Deer	90	28	3.2	0.040	0.100	0.41	....	...	...	219
R	Hillcrest	66	42	1.6	0.015	....	...	....	...	...	227
		18	22	0.8	0.025	0.065	0.37	....	...	...	....
	Average	....	...	...	....	....	...	....	...	...	174
	(d) Unburned Canyons; Normal Zones										
S	Arroyo Seco*	112	100	1.1	0.050	0.032	13.40	....	...	...	8.3
T	Sycamore	42	37	1.1	0.015	0.034	1.80	....	...	...	23.3
U	Stough:										
U-1	North Fork	9	10	0.9	0.030	0.035	0.84	....	...	...	10.7
U-2	East Fork*	8	12	0.7	0.035	0.030	0.58	....	...	...	13.8
V	Tuna	22	18	1.2	0.025	0.016	5.51	....	...	...	4.0
	Average	....	...	...	....	....	...	....	...	...	12.0

* Approximate. † Roughness of channel only. No attempt has been made to compensate for reduced velocities due to debris load. ‡ Values not known.

in which V = velocity in feet per second; n = coefficient of roughness; R = hydraulic radius; and S = hydraulic slope. No conditions are known which would affect its use in these cases. Five of the sections were measured in the zone of normal runoff, as reported in Table 2. All of these measurements were made in canyons which were contiguous to canyons in the zone of excess runoff. The properties of these sections are shown in Table 1(*d*).

TABLE 2.—VELOCITIES AND DISCHARGES COMPILED BY THE MANNING FORMULA; UNBURNED CANYONS; NORMAL ZONE

Sections (see Table 1(*d*))	Drainage area (see Figs. 2 and 3)	Velocity, V, in feet per second	Rate of flow, Q, in cubic feet per second
S	Arroyo Seco	5.7	638
T	Sycamore	23.0	966
U	Stough:		
U-1	North Fork	9.1	82
U-2	East Fork	6.5	51
V	Tuna	8.6	189

The canyons in the normal zone gave runoffs averaging 161 cu ft per sec for each square mile of catchment area. These runoffs are not uncommon. They have been exceeded many times in all parts of the United States.[3] A comparison with them affords a rough idea of the great magnitude of the wave fronts in the zone of excess runoff. They also show what short distances separated canyons in the zone of excess runoff from canyons in the normal zone whose cover and gradients were similar.

The automatic rain gages in this vicinity showed no rainfalls lasting 15 min, at a rate as intense as 2 in. per hr. Even with a very high percentage of runoff this rainfall would not have sustained flow sections greater than from 30 to 40 sq ft per sq mile of catchment area.

The average of measurements made is as follows:

Description	Square feet of flow section per square mile of catchment
In the zone of normal runoff, flow sections averaged	12
In the unburned areas in the zone of excess runoff, flow sections averaged (or 15 times larger than average flows in the normal zone)	174
In the 1927 burn in the zone of excess runoff, flow sections averaged (or 23 times larger than average flows in the normal zone)	283
In the 1933 burn in the zone of excess runoff, after deducting as below for check dams, flow sections averaged (or 57 times larger than the average flows in the normal zone)	689

[3] An excellent "Bibliography Relating to Flood Flow, Intense Rainfall, and Frequencies" is contained in U. S. Geological Survey *Water Supply Paper No. 771*, "Floods in the United States," by Clarence S. Jarvis, M. Am. Soc. C. E., and others, 1936, pp. 468–487.

A runoff at the rate of 1.56 in. in depth each hour over the entire catchment area is required to sustain a flow of 1,000 cu ft per sec for each square mile. A flow of 4,000 cu ft per sec per sq mile requires a runoff at the rate of more than 6 in. in depth each hour, and in these streams would have a section of approximately 130 sq ft for each square mile.

The flow sections in the zone of excess runoff are so large that great care was used to assure that they would be representative of the conditions at the time of the peaks. Many of these measurements were made at locations where there could be no fear that the section presented a distorted situation because of scouring of the bed after the highest elevation of the water surface had occurred.

In the mountains burned in 1933 the measurement in Haines Canyon was made at a concrete U. S. Geological Survey gaging station (see section J, Table 1). Another of the measurements was made on a check dam which remained intact (section E-2, Table 1).

In the zone of excess runoff in the Verdugo Mountains the section which measured the total flow of Brand Canyon was taken in a concrete storm channel. The water had spread into an orchard and some cutting of the ground had occurred, but not much (see section K-1, Table 1). The Allen Canyon flow was measured on an uninjured check dam (section L-1, Table 1). Two of the sections were measured on concrete streets (sections N and R, Table 1). In the zone of normal runoff one of the sections was measured on a concrete street (section T, Table 1). The other sections in this zone showed no signs of scouring.

As to four of the sections in the zone of excess runoff, it could not be said, from indications upon the ground, whether scouring of the bed had occurred after the maximum water elevation was reached (sections C-1, D, I, and P, Table 1). Scouring would be expected to show in the form of sections much larger than the average, but there is nothing to indicate such a situation. Each of the four sections is smaller than the average in its classification. One section showed definite signs of scouring, but gave no clue to the location of the bed of the channel before the flood. It has not been used in computing averages (section C-1, Table 1).

At sections C, E, F, G, and H, Table 1, the remains of small check dams made of wire-bound boulders were present close to the point of measurement. Before the flood these check dams, placed about 50 yd apart, were intact, the space above them largely filled with debris from the canyons. Most of them were swept away by the flood. It is not known definitely whether the check dams were destroyed before or at the time of the surges.

In the absence of information, it is assumed that check dams were intact at the moment of the maximum peak. This assumption necessitates the decrease of such measured sections to allow for the areas that these check dams occupied before their destruction. Remains of check dams showed the extent of scouring of the adjacent channels. At none of the measured sections (except perhaps section C-1, which was not used as a basis for averages) would the check dams, if intact, have reduced the measured area by more than the lowest 4 ft. This

value has been used for the reduction where check dams were located sufficiently near so that they could possibly affect the measurement.

A comparison of the data in Table 1 shows that the average hydraulic properties of the channels in the various classifications tend slightly to equalize the values, rather than to accentuate the differences.

Areas in the Zone of Excess Burned in 1933

Hall-Beckley Canyon.—This canyon lies in the burned part of the Sierra Madre near the eastern limit of the Verdugo Canyon basin. At a point in the canyon where the catchment area is 0.67 sq mile, the trees are still completely scoured of bark on their upstream side to a height of many feet above the stream bed. The appearance and shape of the channel at this point are shown in Fig. 4, taken more than three years after the flood. This view was

Fig. 4.—Height of Scour in Hall-Beckley Canyon

taken at one of the few remaining places which still give an idea of the magnitude of these surges. The stream bed is 5 ft below the bank on which the tree is growing. About 2,000 ft downstream from this point a measurement was made in a straight section where marks of the water surface were well defined. The catchment area above this section measured 0.72 sq mile. The flow section was 95 ft wide at the surface and almost 13 ft deep to the bed of the channel. After deducting the lowest 4 ft for the area occupied by check dams before the storm, the flow section was 680 sq ft, or at the rate of 944 ft for each square mile of catchment area. The slope was 0.069. Indications of water surface were unmistakable. At the height adopted, the bark was completely scoured from the upstream face of an oak tree 9 ft from the left bank. Splash from the water extended from 10 to 12 ft higher into the tree, equivalent to the velocity

head at about 25 ft per sec. There could be no doubt of either the water surface or the high velocity.

Where the canyon joins the debris cone it has an area of 0.86 sq mile. At this point it is met by the channel from Snover and Weber canyons and 0.56 sq mile is added to the area. Below the confluence the flow enlarged the channel, which previously had been about 12 ft wide and 4 ft deep, until it was more than 50 ft wide and 6 ft deep. Notwithstanding this increase in area, the channel was so inadequate to carry the water that at its maximum flow the water was 5 ft deep in the yard of the residence shown in Fig. 5 and drowned two people there who had been clinging to a truck. This water included all of the flow from the Hall-Beckley Canyon, but only part of the flow from the Snover area. Testimony of many witnesses in court showed that the channel was of this size before the occurrence of the surge.

FIG. 5.—HALL-BECKLEY CHANNEL BELOW CONFLUENCE WITH SNOVER CHANNEL

Pickens Canyon.—This canyon also lies in the fire-swept Sierra Madre, about 1 mile west of Hall-Beckley Canyon. It heads on the ridge 3,300 ft east of Mount Sister Elsie, 5,081 ft high, at whose crest an automatic rain gage was placed. Flood heights were measured in this stream at six places, four of which were in the deep channel downstream from the canyon and included the entire flow.

A short distance downstream from the lowest tributary the flow section above the tops of the check dams was 621 sq ft. The water surface was 53 ft wide. At this point the channel curves through an arc of 180° with a radius of about 150 ft. Marks of the water surface were well defined and unmistakable. The super-elevation at the section was 6.75 ft; and the slope was 0.087 (see section C-3, Table 1). Downstream 60 ft the water had overtopped heavily, and eroded to a depth of 4 ft a bank of semi-consolidated gravel which is 22 ft higher than the tops of check dams. At this point the channel was 45 ft wide 20 ft below the top of the eroded bank. The catchment area at this point is 1.78 mile. Three-fourths of a mile downstream, at Mountain Street, the water

was barely confined in the channel. The flow section above check dam tops (above the highest part of the check dams) was 590 sq ft (section C-5, Table 1); 100 ft downstream the water had overtopped the banks so heavily that a boulder 8 ft long, and a part of a check dam 4 ft by 4 ft by 18 ft long had been thrown entirely out of the channel, and within 20 ft of an unoccupied residence.

For one-half mile downstream from Foothill Boulevard the channel had a cross section about as shown in Fig. 6. This section of channel was much too small to carry the flow. The water overflowed so deeply at Evelyn Street that it completely scoured the bark from the upstream side of a Eucalyptus tree to a

FIG. 6.—VIEW OF RESIDENCE CLOSE TO PICKENS CHANNEL AFTER THE STORM OF JANUARY 1, 1934

height of 7 ft above the ground. This tree is 50 ft west from the channel. On the east side of the channel, downstream, many boulders from 8 to 10 ft long were thrown out of the stream bed. The catchment area at this point is 1.82 miles. Measurements made by the Los Angeles County Flood Control District were of similar magnitude. Five sections in a distance of 140 ft gave an average flow section of 600 sq ft.[4]

Other Canyons.—The neighboring canyons in the Sierra Madre emitted similar surges, some of them of greater intensity than the foregoing. Drainage areas, flow sections, and hydraulic properties are shown in Table 1.

VERDUGO MOUNTAINS

The surges from the Verdugo Mountains, parallel to and about 4 miles south of the Sierra Madre, were smaller but were still excessive. They showed the response of the runoff to a variety of conditions.

A part of the Verdugo Mountains had been burned over in 1927 and vegetation had only partly recovered. The fire had denuded canyons within the zone of excess runoff as well as canyons within the normal zone. The mountain

[4] "Los Angeles County Flood Control District Report New Years Foothill Debris Flood, 1934," by E. C. Eaton, M. Am. Soc. C. E., report dated March, 1934, Chart No. 4 (not published).

fires do not ordinarily destroy the larger trees, or the roots of chaparral, and the cover reestablishes itself faster than would be expected. The natural covering is a chapparral composed of various species of native brush, from 6 to 10 ft high. This covering breaks the direct force of a heavy rain and keeps the ground beneath covered with leaves and mulch, with a tendency both to decrease and to retard runoff.

Brand Canyon was one of the burned regions in the zone of excess runoff. It has an area of 1.06 sq miles, and lies on the south slope of the Verdugo Mountains. In this canyon a rock 87 in. long was thrown entirely out of the natural stream bed, and was lifted into a three-pronged tree near the channel. It rested there 4 ft above the stream bed. At the mouth of the canyon the flow section was 233 sq ft. The canyons adjoining Brand Canyon on the east were unburned, and vegetation was normal. Hillcrest Canyon is contiguous to Brand Canyon. Its drainage area is 0.37 mile. It discharges into a concrete street 40 ft wide. It was one of the sections best adapted to accurate measurement. No doubt existed as to the flow section, and the value of n could be approximated closely. The section was 84 sq ft, or at the rate of 227 ft for each square mile of catchment area (section R, Table 1). This section would have carried 2,500 cu ft per sec of clear water. Deer Canyon, also unburned, and contiguous to Brand Canyon, gave a flow section of 90 sq ft from a drainage area of 0.41 sq mile, or at the rate of 219 sq ft per mile of catchment area (section R, Table 1). The flow sections from other unburned canyons in the zone of excess runoff are shown in Table 1(*c*).

The western part of the Verdugo Mountains was not in the zone of excess runoff. Sunset Canyon is west of Brand Canyon. The two canyons were burned over at the same time. Debris carried down by the flood was impounded by a small dam in Sunset Canyon. This dam caught about 200 yd of debris from an area of 0.44 sq mile. This was at the rate of approximately 500 cu yd per sq mile. Below Brand Canyon, which was in the zone of excess runoff, 75,000 cu yd of debris per sq mile of catchment area were removed.

Rainfall on January 1, 1934.—The rainfall which caused these surges was a short violent storm of highly uncommon characteristics. At Los Angeles, about 12 miles south of the destructive area, the automatic gage of the U. S. Weather Bureau caught 7.36 in. in the wettest 24 hr. Until March, 1938, this 62-yr record showed no other period as short as 3.5 days with an equal rainfall.

Near the mountains, the gages show a surprising uniformity of rainfall over a large area, almost regardless of great differences in elevation, topography, location, and exposure. In the zone of excess runoff, and on the valley floors to the south, an average for the storm of 12.93 in. fell on an area of 80 sq miles. Approximately 12 in. of this fell in the wettest 24 hr. Almost all of the sixteen gages in the zone of excess were in the rugged mountains, at an average elevation of 2,100 ft. They showed a catch differing by 1% from the catch of the sixteen gages to the south, at an average elevation of 831 ft. The aforementioned thirty-two gages varied in elevation from 526 to 5,081 ft. This area of nearly uniform rainfall included the entire zone of excess runoff. The readings of the gages are summarized as follows:

Description	Number of gages	Rainfall for storm, in inches
Zone of Excess:		
All gages	16	12.86
Gages in 1933 burn	2	11.52
Gages close to edge of burn	5	12.02
Zone of Normal Runoff:		
Belt south of zone of excess	16	12.99
Arroyo Seco catchment east of zone of excess	5	12.12

The slight influence of elevation on the rainfall in this area during the storm on New Year's day is shown by tabulating rainfalls in the order of elevation of the gage. Placed in this order they show:

Elevtion of gages, in feet	Number of gages	Rainfall for storm, in inches
Below 1,000	11	13.46
Between 1,000 and 2,000	15	12.84
Between 2,000 and 3,000	7	12.15
Between 3,000 and 4,000	3	12.07
Above 4,000	2	12.15

The automatic gages in Haines Canyon, and on Mount Sister Elsie in the zone of excess runoff, reached their capacity before the end of the storm. They did not record its later stages, although they gave a good record up to, and for some time after, the surges. They also showed the total amount of rainfall for the storm. The automatic gage at Flintridge, from 2 to 6 miles distant from the canyons which emitted surges, was the nearest automatic gage that recorded the entire storm. It caught a total of 14.02 in., of which more than 99% fell in 40 hr, and 12.86 in., or 92%, fell in the wettest 24 hr. The gage at the Flintridge fire station is about 1 mile east of the Verdugo Canyon catchment area. It is 6 miles southeast from the gage on Mount Sister Elsie, on the edge of the area at an elevation of 5,081 ft. The Haines Canyon gage is in the center of Haines Canyon, less than a mile from that on Mount Sister Elsie, but more than 1,600 ft lower. The two latter gages are in the burned area in the mountains. Mount Sister Elsie is at the heads of Haines and Dunsmuir canyons, and within 1 mile of the heads of Pickens, Shields, and Blanchard canyons. The head of Cooks Canyon is 2,000 ft distant.

There were no noteworthy short-time rainfall intensities. One gage, several miles from the zone of excess, showed rainfall of more than 0.5 in. in 15 min. The gages within and close to the burned area showed the rainfall rates given in Table 3. The automatic gages at the Polytechnic High School and at the Weather Bureau in Los Angles, directly south from the zone of excess runoff, showed similar abrupt increases in rainfall.

The automatic gage in Brand Canyon reached its capacity before the surge, but the following notes of the observer at the mouth of the canyon supply much of the missing data: "Unusual down pour at 11:50 P.M. Dec. 31. Runoff

reached flood proportions at 12:05 A.M. Jan. 1, at which time a sudden rise in volume of stream caused it to leave its channel ***." These increases, falling upon steep, thoroughly soaked, freshly burned canyons, produced the afore-

TABLE 3.—STUDY OF SHORT-TIME RAINFALL INTENSITIES

Locality	Elevation, in feet	Time of increase	Rate immediately before increase	Rate immediately after decrease	Rate of increase
Haines Canyon	3,470	11:37 p.m.	0.60	1.56	0.96
Mount Sister Elsie	5,081	11:45 p.m.	0.60	1.56	0.96
Flintridge	1,325	12:01 a.m.	0.24	1.08	0.84
Average	....	....	0.48	1.40	0.92

mentioned surges. Table 3 shows the moderate rates of rainfall sufficient to cause such violent surges when other conditions are favorable for their creation. All of the surges followed promptly after the increase of rainfall. This was true not only in this storm, but also in all of the other storms noted herein. Rainfall increases during the other Southern California storms listed herein were similar. Some of the intensities are shown in Fig. 7.

Other Southern California Surges

Several less important floods of the same type have occurred in Southern California since automatic rain gages have been installed in the mountains. The first of these came on November 14, 1928, from Allen, Brand, and some small adjacent canyons burned over in December, 1927. Practically all of this area except Sunset Canyon was in the zone of excess runoff on January 1, 1934. Except in Sunset Canyon, peak intensities seem to have been not greatly different from the 1934 surges, but the entire duration of the flow was not greater than 20 min.

In Allen Canyon four measurements by the Flood Control District gave an average flow section from 0.2 mile of 95.25 sq ft. This was at the rate of 476 sq ft of section per square mile of catchment area. In Brand Canyon the surge was stated to have been higher than that of 1934.

The rainfall which caused these surges was measured by sixteen gages within 5 miles of the zone of excess runoff. None showed a total catch greater than 1.45 in. The two automatic gages in the vicinity showed a maximum 10-min rate of 1.62 in. per hr, although the maximum 5-min rate was 2.52 in. per hr. In February, 1929, Brand Canyon emitted a much smaller surge during a rain which would almost pass unnoticed under ordinary conditions. The automatic gage then in the canyon showed a rainfall starting suddenly, with a maximum rate of 1.80 in. per hr. The entire rainfall lasted only 15 min, and the total rain on the 1.07 sq mile was less than 12 acre-ft.

In October, and again in December of 1934, the then recently burned area in the Sierra Madre yielded surges much smaller than those of January 1. The automatic gage in Haines Canyon showed a maximum rainfall rate of 1.68 in. per hr in December. In October this gage showed a 5-min rain at the rate of 3.5 in. per hr.

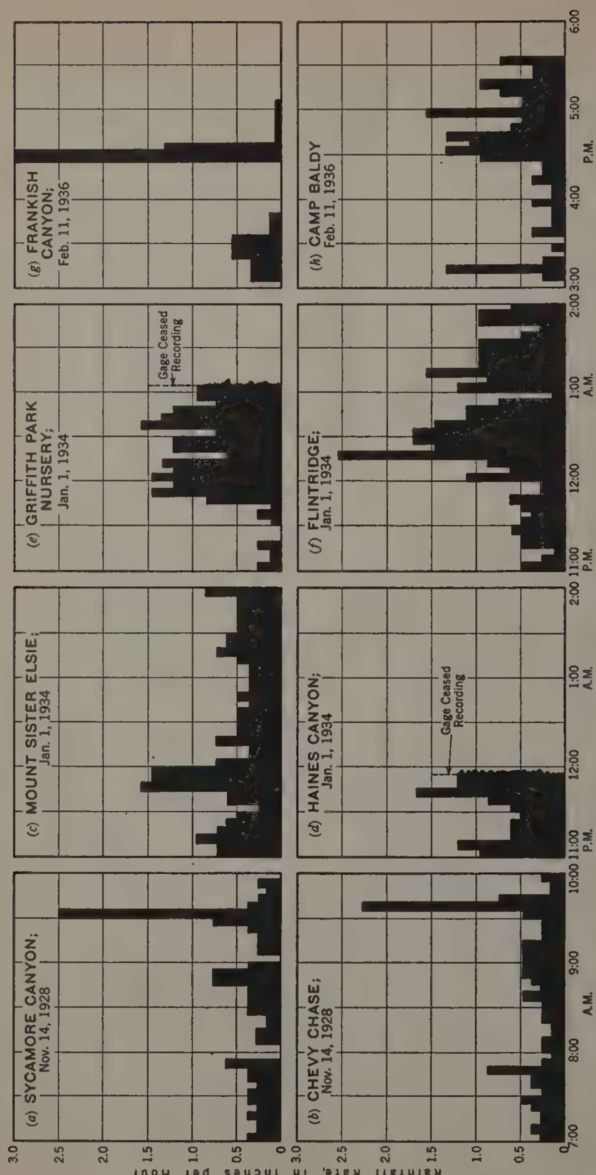

FIG. 7.—RATE OF RAINFALL FOR EACH 5-MIN PERIOD DURING THE TIME CONSIDERED

A smaller flood of this character occurred about 30 miles to the east on the south slope of the Sierra Madre on February 11, 1936. This flow came from a then recently burned canyon (burned in 1935) with an area of 0.56 sq mile. The surge was estimated by witnesses to be 10 ft high. None of the neighboring unburned canyons yielded unusual runoffs.

This flow was caused by a sudden rain beginning on recently burned, wet canyon slopes. For 7 min rain fell at a rate of 3.00 in. per hr. It then dropped for 5 min to 1.32 in. per hr, and then to a light sprinkle. During the first 7 min less than 10 acre-ft of rain fell on the entire canyon. The total volume of rain for the 12 min was less than 13 acre-ft, which fell at a maximum rate of 1,000 cu ft of water per sec.

Other Floods.—Engineering publications contain accounts of many floods in the United States in which, apparently, the runoff peak cannot be expressed as a percentage of the rainfall on the area. In these descriptions are included more than forty floods in which peak flows have been measured at quantities ranging from 1,000 to 4,170 cu ft per sec for each square mile of catchment area. More than one half of these runoff peaks have been reported from east of the Mississippi River. In some of these cases, the absence of automatic rain gages makes it uncertain whether the runoff peak was caused by an accumulated surge, or as the direct result of an excessive rate of rainfall for a period longer than the concentration time of the area. It is difficult or impossible to reconcile many of these runoff figures with the rainfall measurements. Some of the accounts seem to show definitely that the conditions necessary to an accumulated peak were present. It is possible that a reexamination of the data would show these conditions in other cases.

Only three of the floods which have been reported, and which probably are of this character, are listed herein (Cardens Bluff, Tenn., and Northern Utah in 1923 and in 1930), although many of the other floods reported seem equally to belong in this classification.

Cardens Bluff.—The flood on the Watauga River, in eastern Tennessee on June 13, 1924, is the first.[5] At this place 14.98 in. of rain fell in about 8 hr. About four fifths of this rain fell between 6:30 p.m. and 10:00 p.m. The rain then decreased, but did not stop entirely until 12:30 a.m., when a second hard rain began that lasted for 2 hr. A small ravine, whose catchment area is about 0.025 sq mile, emitted a surge 8 to 10 ft in height which totally demolished two houses and killed nine of the occupants.

Northern Utah.—Several floods which originated in the mountains north of Salt Lake City, Utah, in 1923 and in 1930 have been the subject of investigation and report.[6, 7, 8, 9, 10] The first of these floods occurred on August 13, 1923,

[5] "Record Cloudburst Flood in Carter County, Tenn., June 13, 1924," by Warren R. King, *Monthly Weather Review*, June, 1924, p. 311; also "Surface Waters of Tennessee," by Warren R. King, State of Tennessee, Division of Geology, *Bulletin No. 40*, 1931, p. 61.

[6] "The Floods of 1923 in Northern Utah," by J. H. Paul and F. S. Baker, *Bulletin* of the Univ. of Utah, Vol. XV, No. 3, March, 1923.

[7] "Floods at Farmington and Willard, Utah," by J. Cecil Alter, *Monthly Weather Review*, Vol. 51, August, 1923, p. 420.

[8] "Torrential Floods in Northern Utah," by a special commission, S. Q. Cannon, M. Am. Soc. C. E., Chairman, Agricultural Experiment Station, Utah State Agricultural Coll., Logan, Utah, *Circular No. 92*, January, 1931.

[9] "Floods and Accelerated Erosion in Northern Utah," by Reed W. Bailey, C. L. Forsling, and R. J. Becraft, U. S. Dept. of Agriculture, *Miscellaneous Publication No. 196*, August, 1934.

[10] "Mud Floods in Utah," by J. Cecil Alter, *Monthly Weather Review*, Vol. 58, August, 1930, p. 319.

in Farmington, Willard, and other canyons along the steep western front of the Wasatch Mountains facing Great Salt Lake. Late in the afternoon of that day an exceedingly abrupt, heavy rain brought enormous, short-time flows from several of the canyons, destroying nine lives and much property. At Farmington the flood arrived within 20 min of the beginning of the rain at that place, doing great damage. At Willard Canyon, the discharge was even greater and more destructive. The canyons are steep, and largely denuded by overgrazing.

The storm that caused these floods was a short, sharp shower, which yielded less than 1.5 in. of rain. At Salt Lake City, 16 miles south, the rainfall was the most intense on record. Most of the rain fell in 15 min at an average rate of 3.04 in. per hr.

Floods of 1930 in Utah.—In July and August, 1930, the same district was again swept by short-lived flows which swept boulders weighing 200 and 300 tons, together with smaller rocks, out of the canyons. These floods were even more severe than that of 1923. The fronts of the waves were reported to be several feet high and were not preceded by a flow of water. The storm of August 11, 1930, was also a short downpour. Ogden received 1.72 in. of rain, 0.40 in. falling in 9 min. This was at the rate of 2.67 in. per hr.

Recapitulation

The various Southern California surges permit of quantitative comparison because of the similarity of some of their conditions. In each of the cases listed, the flood has occurred in a steep canyon, after an abrupt increase of rainfall. For the purposes of comparison, the severity of the surges has been placed in four classifications (see Table 4). In classifying these surges, account has been taken, not only of the flow sections, but also of the properties of the channels, and of the comparative destructiveness of the flows after they left the canyon mouths.

TABLE 4.—Comparative Severity of Surges

Locality	Date	Time	Condition	Age of burn, in months	Slope drop in first mile, in feet	Severity
Brand-Allen-Sunset	11/14/28	9:25 a.m.	Burned	11	1,700	Heavy
Brand	4/ 4/29	9:40 a.m.	2/3 burned	16	1,700	Light
Sierra Madre	1/ 1/34	12:01 a.m.	Burned	1.5	1,700	Violent
Brand	1/ 1/34	12:01 a.m.	2/3 burned	72	1,700	Heavy
Hillcrest-Deer	1/ 1/34	12:01 a.m.	Not burned	....	1,700	Moderate
Baldridge-Whiting	1/ 1/34	12:01 a.m.	Not burned	....	1,700	Light
Sierra Madre	10/17/34	4:00 p.m.	Burned	11	1,700	Light
Sierra Madre	10/18/34	2:00 a.m.	Burned	11	1,700	Moderate
Fern	10/18/34	2:00 a.m.	Burned	3	1,700	Heavy
Sierra Madre	12/14/34	7:00 a.m.	Burned	13	1,700	Light
Frankish	2/11/36	4:45 p.m.	Burned	6	1,700	Heavy
Sycamore	1/ 1/34	12:01 a.m.	Not burned	....	865	None

Conclusions

In searching the evidence disclosed by these surges, many factors emerge as having a strong bearing on their occurrence and size. Some of the factors involved can be evaluated roughly, but several of them can only be recognized as important, though unmeasured, items in the final result.

The combination of conditions necessary to the formation of the surges is not common. They are not especially a problem of the climatic and topographic environment of the west. Whenever rain increases abruptly to a sufficient intensity upon steep slopes, the formation of the surges may be expected. Small, denuded areas facilitate their creation. When all of these conditions are present, a torrential rainfall is not required. If any of the conditions are missing, or are present to a diminished degree, the surges are reduced in size or are suppressed entirely. On the other hand, a torrential rainfall causes them even though other conditions are not favorable. When conditions are favorable, one of the most striking features of these floods is the disparity between the magnitude of the surges and the rate of rainfall at the time of their occurrence.

When the effects of the storm of January 1, 1934, were studied both in the Sierra Madre and in the Verdugo Mountains, four facts stood out with special prominence:

(1) The unexpectedly large flow sections;

(2) The lack of conformity between the various burns and the zone of excess;

(3) The nearly simultaneous formation of the surges in the various canyons; and

(4) The countless evidences of high velocity.

The first of these has already been discussed. The channels shown in Figs. 4 and 5 apparently were not half large enough to carry the flow.

Lack of Conformity with "Burns"

Lack of conformity between the area burned in 1933 and the zone of excess runoff was not noted in the Sierra Madre Mountains; but in the Verdugo Mountains the area burned in 1927 apparently did not coincide at any point with the zone of excess runoff.

The surges occurred in all of the canyons investigated in a total area of more than 10 sq miles. They did not form in some burned canyons outside of this area, but did form in other nonburned canyons within the area. Brand Canyon and Sunset Canyon were both in the 1927 "burn." Sunset Canyon was not in the zone of excess runoff in the storm of January 1, 1934.

The lack of conformity also extended to nonburned areas. Attention has already been called to the surges from the nonburned part of the Verdugo Mountains. Within the zone of excess, these canyons discharged quantities of debris only slightly less than from the burned areas. Below Hillcrest Canyon, and the smaller canyon to the east, both not burned, the debris removed was at the rate of more than 60,000 cu yd for each square mile of catchment area (see section R, Table 1). The evidence on the ground seemed to show clearly that the "burn" was an exceedingly important item, but that it was not the only, or even an essential, factor in the formation of the surges.

Nearly Simultaneous Formation of the Surges

The "nearly simultaneous formation of the surges" is interpreted herein as indicating definitely that the cause of the surges was the change in rainfall

conditions. No other factor seems to account for their formation at this particular time, in all of the canyons within this comparatively wide area, without regard to differences in the size of the various canyons, in elevation, or in the condition of the vegetation.

The surges occurred in all of the cases investigated soon after the abrupt increase of rainfall, and not at the culmination. In this respect, as in many others, they differ from the usual type of flood, which comes to a peak at a much later stage of the rainfall.

Velocity

There were many indications of the surface velocity. Splash marks from 10 to 14 ft above high-water level were noted and photographed at almost all points along the natural channels. Near the lower end of the Pickens Channel where the erosion and destructiveness had decreased, the flow had piled rocks and sand on to a roof which stood nearly in the path of highest velocity. The rocks were lodged 10 ft above the high-water mark, and had been thrown forward at least 11 ft. Below Cooks Canyon a boulder had been thrown into a tree 8 ft above high-water marks. Its horizontal movement was not less than 10 ft.

This material left the water surface at velocities of 25 to 30 ft per sec. Apparently, surface velocities were not much lower than would be indicated by the Manning formula. Velocities in the lower part of the stream are conjectural. The flows were carrying very heavy percentages of debris. The extent to which the bed load slowed this part of the flow is not known.

Condition of Surface

In a canyon denuded of vegetation, the response of the stream to changing rainfall conditions is greatly accelerated, and increased rainfall is translated into increased runoff much more quickly than is the case where the ground is covered with vegetation.

Although a denuded surface invites the formation of these surges, they form, under conditions favoring them, on semi-denuded, or even on completely covered areas.

Hillcrest, Deer, Baldridge, and Whiting canyons were fully covered on January 1, 1934, with the normal chaparral growth. The brush cover increases in density from Hillcrest to Whiting Canyon, and the surges from these canyons showed a steady decrease from Hillcrest to Whiting Canyon.

The floods of 1924, in eastern Tennessee, occurred in a densely wooded environment, but rainfall rates were so high that the cover did not give sufficient protection, and very violent surges occurred.

Brand Canyon was burned in 1927. At the time of the flood on New Year's day, 1934, this vegetation had recovered during a 6-yr period. Many photographs of this canyon taken during the investigation following the New Year's storm seem to indicate that the brush had recovered about one half of its normal protective effect. During that storm, the surge from Brand Canyon was much smaller than from the then recently burned canyons.

Steepness of Canyons

The important part played by the steep slope is illustrated by Sycamore Canyon, 1 mile east of the nonburned canyons in the Verdugo Mountains. Gradients in Sycamore Canyon are much lower than in the zone of excess runoff. This is true, not only of the stream bed, but of the side slopes as well.

So far as noted, the lower gradient was the only respect in which Sycamore Canyon was less favorable to the formation of the surges than the four nonburned canyons in the Verdugo Mountains. Three rain gages showed the rainfall of the storm of January 1, 1934, to be somewhat greater than in the canyons which emitted surges. One of these was an automatic gage and showed that rainfall characteristics were similar. The brush covering was complete, but not so dense as in the nonburned canyons in the Verdugo Mountains.

The entire runoff from this canyon flowed down a concrete street. The runoff peak, per square mile of catchment area, was not more than one tenth as great as the peak from the nonburned Hillcrest Canyon, also measured in a concrete street.

Duration of Peaks

Various evidence upon the ground, as well as statements of many witnesses, showed that maximum peaks in the flood of January 1, 1934, were of short duration.

In Haines Canyon, a reliable approximation can be made of the total material in the wave front. At the mouth of the canyon, about one fourth of a mile downstream from the concrete gaging station, an old gravel pit had been utilized by the Flood Control District so as to make a suitable debris basin. It had an area at the spillway level of 2.13 acres. Throughout the evening of December 31, the debris basin was observed by a watchman of the adjacent rock crushing plant, and also by an employee of the Flood Control District who was detailed to watch the water level in the basin. Just before the surge water level was from 4 to 5 ft below the spillway. The surge raised the level from that point until it flowed 2 ft deep over the spillway. The indications are that the surge in Haines Canyon contained from 15 to 17 acre-ft before the flow dropped to a point where it was carried by the spillway without raising the level in the basin. This value includes water and debris, but not entrained air (section J, Table 1).

Debris Content

The surges were very heavily charged with debris. Most of the witnesses who placed a value on the debris content estimated the flow as 50% water and 50% solids. Three observers noted the character of the flow on the streets below Hall-Beckley Canyon. One of them estimated it as 50% solids, whereas another who had seen the same flow at the same time estimated it at 60% solids. E. C. Eaton, M. Am. Soc. C. E., who saw the flow near this point, estimated the solids as from 50% to 70% of the total.[11] The flow from Shields Canyon was estimated by one observer to be 50% solids, and to be "definitely thicker than water."

[11] "Los Angeles County Flood Control District Report New Years Foothills Debris Flood, 1934," by E. C. Eaton, report dated March, 1934, p. 17-C (not published).

In Deer Canyon an observer stated that the flow "wasn't like water * * * like a thin mixture of cement with rocks." Another observer said there were "plenty of rocks on the bottom but the flow itself was distinctly 'water.'" In Pickens Canyon the flow was stated to be "very muddy but splashing."

Acknowledgment

Indebtedness in the collection of data, or in the preparation of this paper, is acknowledged to Edward M. Lynch, to Charles G. Frisbie, and to Allen Van Rensselaer. Mr. Frisbie joined with the writer, especially, in making the measurements recorded in Table 1. Much valuable assistance has also been given by the Los Angeles County Flood Control District, the California Forest and Range Experiment Station of the U. S. Department of Agriculture, the U. S. Weather Bureau, the U. S. Geological Survey, as well as by many others.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TRANSPORTATION DEVELOPMENTS IN THE UNITED STATES

Discussion

BY FRED LAVIS, M. AM. SOC. C. E.

FRED LAVIS,[36] M. AM. SOC. C. E. (by letter).[36a]—The discussions of the paper have been very interesting and amply confirm the contention of the writer that the railway problem in the United States is one of national and very vital importance. Some of the commentators seem to have overlooked the fact that this paper was originally presented as the opening address in the "Symposium on Transportation" offered at the Annual Convention of the Society in July of 1938, and seem to have assumed that it was a voluntary defense of the railroads rather than, as it really was, an attempt to evaluate the position of the railroads as part of the general scheme or plan of transportation in the United States.

Professor Cunningham notes that although a statement is made in the "Synopsis" that "Special stress is laid on the need for adequate research," little is said about it in the paper. This matter is referred to in more detail by Mr. Budd.

While most of the commentators agree as to the nature of the problem and its seriousness they disagree as to its solution or the remedies to be applied. Mr. Harris' indictment of present-day management, although he admits that "it is responsible for the finest system of railroads in the world," ignores a great many factors which are of vital importance to the nation in considering the problem. The greatest of these is the tyranny of railway labor and its influence on legislation. Reference was made to this in the paper but further reference may be made to a statement by Garet Garrett[37] which, while crediting the railway labor unions with a most creditable record as workmen, shows the abuses

NOTE.—This paper by Fred Lavis, M. Am. Soc. C. E., was presented at the Annual Convention, Salt Lake City, Utah, on July 20, 1938, as part of the Symposium on Transportation, and was published in November, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1939, by Messrs. Milton Harris, and Ralph Budd; and March, 1939, by Messrs. L. Alfred Jenny, William J. Cunningham, S. R. Truesdell, David A. Molitor, and C. A. Hoglund.

[36] Cons. Engr., New York, N. Y.

[36a] Received by the Secretary September 21, 1939.

[37] "Peace on the Rails," by Garet Garrett, *The Saturday Evening Post*, September 9, 1939, p. 8.

which have crept in and become established in the "constructive" rulings and in the rulings of the Railway Mediation Board.

Referring to railway and highway financing, Mr. Harris states that the main difference between them is that "The highways are not listed on the Stock Exchange." However, he ignores the fact that the Government bonds which paid for them are listed; that the capital of the banks and of the insurance companies is tied up in these bonds; that the stocks of the motor vehicle corporations are listed; and that they, with the labor organizations which are occupied in the motor vehicle industry, exercise a great deal of influence with the various State Legislatures as well as with the Congress. These factors cannot be ignored; nor can one, in facing the problems of to-day and trying to find a solution for them, get very far by blaming the financiers of yesterday.

No one realizes better than the writer the sins of railway financing in the past; but the securities issued are now in the hands of a large number of people in the United States, either directly or through the insurance policies so many of them hold. The problem is not one for the relief of Wall Street but for the relief of a very large number of the people of the United States who are not banded together and are not vocal. Above all, however, it is a problem of how to preserve a great asset of the country—one on which so many industries must depend.

Professor Cunningham points out that while it may be, or probably is, true that the railways are not over-capitalized, from the viewpoint of dollars invested, they probably are from the viewpoint of "dollars invested in facilities that are now not used or useful." Perhaps there is some little merit in this contention but it ignores the many improvements out of earnings and the more intensive use (under normal conditions) of the facilities which remain.

Mr. Jenny also argues that the fact that the railroads do not earn enough to pay a fair return on their outstanding securities is evidence that they are over-capitalized. This fact, however, does not seem to be valid in view of what is said in the paper and the discussions as to the difficulties of modernizing their plant, of conducting reseach, and of being ground between the upper and lower millstones of Government regulation and labor union demands. Whether railroads are over-capitalized or not, the problem facing the nation is the rehabilitation of the transportation machine to produce ton-miles and passenger-miles efficiently and reliably at the lowest possible costs.

Perhaps the statement in the paper that Diesel engines (locomotives) have not yet progressed "much beyond the experimental stage for general railroad traction" may be subject to misinterpretation. That they are actually performing important services, with reliability, day in and day out, cannot be questioned; but in the original statement it was intended to call attention to the fact that, as yet, there is not much information at hand about final costs of service, and that Diesel engines are being used now for only a very small part of the total railway traction requirements.

Mr. Budd's remarks on the rate question, or some parts of the rate questions, are very much to the point and answer a good part of Mr. Harris' criticism of management—that is, of financial management. In times of stress, the rail-

ways are not only faced by demands for higher wages, supported by, and to all intents and purposes practically enforced by, the National Government, but are told by another Government agency what they may charge for service.

The writer is not for a moment advocating higher rates generally on railways, although some may be justified. Rather he is emphasizing that the greatest transportation agency in the United States, an agency vital to the existence of the country, is falling between the demands for higher wages, lower costs of transportation, and competition by Government financed agencies.

In the opinion of the writer, after studying the very interesting and able discussions on this subject which have appeared, it is more than ever evident that the only solution of the problem is the entire modernization of the railroad plant so that the lower costs, and therefore rates, can be produced and reasonably good wages paid. There can be little quarrel with reasonable, even something more than ordinary, compensation for railroad workers. They must have a high mental and moral standing as well as requisite skill; but many of the anomalies of "constructive" rulings should be abolished and reasonable disciplinary measures should be upheld.

No matter what form the assistance may take, the writer can see no alternative to such a subsidy to the railways as will permit this necessary modernization and the establishment of adequate research facilities on a very broad scale.

There certainly can be no more objection to this, and perhaps much more reason for it, than for the subsidies being generously donated to waterways, aviation, and highways. As for burdening the railways with debt for loans to be made, why should this be any more necessary than it has been for the construction of the other and competing facilities without placing on them any burden?

Corrections for *Transactions: Proceedings*, March, 1939, page 488, line 39 should read "*** making of Government loans to carriers; and, he states ***"; in the caption for Table 4(*a*) change "per Operated Ton-Mile" to "per Equated Ton-Mile"; and, to each of the values in Table 5, Columns (2), (3), (4), and (5), add three zeros.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FLASH-BOARD PINS

Discussion

By Messrs. William P. Creager, Lincoln W. Ryder, and E. T. Schuleen

William P. Creager,[6] M. Am. Soc. C. E. (by letter).[6a]—An excellent series of experiments, indicating the strength of flash-board pins, has been presented in this paper. It should be noted that all of the experiments were made with a completely aerated jet yet, in most cases in practice, the jet is not aerated and a vacuum load is added to the pins.

The designer aims to provide an installation which will surely fail before the water surface reaches an undesirable elevation. On the other hand, he does not want failure to occur too soon because that would result in a greater number of replacements. Therefore, he is anxious to design the pins with as great an accuracy as possible.

The authors recommend a modulus of rupture varying from 70 000 lb per sq in. for 3-in. and 2-in. pipe to 80 000 for $\frac{3}{4}$-in. pipe. The writer found, by tests[7] made prior to 1926, that a modulus of rupture between 42 000 and 58 000 lb per sq in. applied to a number of actual installations utilizing 2-in. to 3-in. pipes. The difference between these stresses and those found by the authors may be attributed to three factors:

(1) The tests made by the writer involved actual cases in which a vacuum occurred under the jet. The amount of the vacuum being unknown, the computed failure stresses of 42 000 to 58 000 lb per sq in. disregarded the vacuum. If this vacuum had not occurred, as in the case of the authors' experiments, the computed failure stress would have been higher.

(2) Present-day manufacture of such pipes requires a descaling operation which involves rolling the pipe in the finishing passes at lower temperatures, which has a tendency to raise the yield point.

Note.—This paper by Chilton A. Wright and Clifford A. Betts, Members, Am. Soc. C. E., was published in May, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[6] Cons. Engr., Buffalo, N. Y.

[6a] Received by the Secretary July 10, 1939.

[7] "Hydro-Electric Handbook," by William P. Creager and Joel D. Justin, Members, Am. Soc. C. E., Art. 132, p. 263.

(3) Present-day tolerances in pipe thickness are "tighter" than they were formerly. Hence, the average pipe thickness to-day is greater, resulting in greater bending strength.

If water higher than that anticipated would cause damage, the writer recommends the use of the authors' design stresses for the first installation. In all probability the effect of vacuum in the usual clear crest installation will result in the boards failing too soon and the use of somewhat stronger pins in succeeding installations.

LINCOLN W. RYDER,[8] JUN. AM. SOC. C. E. (by letter).[8a]—In August, 1939, in connection with the design and construction of three small flash-board dams for gound-water level control on an open drainage channel, it was necessary to select pins that would fail within a small range in the height of water above the crest of the dam. It was required that the boards remain in place with 1 ft, and certainly fail with 2 ft, of water overtopping them. The purpose of the tests was to determine the maximum bending moments of the pipe specimens chosen tentatively by the designing engineer[9] in order to check and control the variation in overtopping to be expected in the field.

The apparatus, set up as shown in Fig. 13, differed from the straight cantilever principle utilized by the authors in having two specimens of the same size

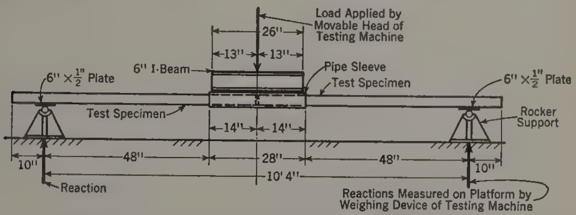

FIG. 13.—METHOD OF TESTING FLASH-BOARD PINS

inserted into opposite ends of a close-fitting pipe sleeve. The load was applied at the middle of the sleeve through a stiff section of I-beam. The sleeve and stiff member simulated closely the action of the cantilever jig in the aforementioned test, there being no appreciable distortion in the sleeve. It is believed that this test rig is simpler than that used by the authors and will yield equally correct results.

The low height of the end supports would not allow the bending in the test of the pins to the greater than 20° amount possible in the authors' test; but modifications in this set-up, supporting the ends of the pins farther above the platform of the machine, would allow bends, limited only by the vertical travel of the screw or the ultimate breaking point of the pipe specimen. Such a modification is not necessary for the test as conducted, however, since the maximum bending moment was reached before a bend greater than 8° was made.

[8] Asst. Engr., Metcalf & Eddy, Boston, Mass.

[8a] Received by the Secretary September 14, 1939.

[9] "Hydro-Electric Handbook," by William P. Creager and Joel D. Justin, Members, Am. Soc. C. E., p. 265.

The specimens for the first test were 6-ft lengths of 2-in., nominal diameter, standard weight, seamless steel tubing marked "Pittsburgh-P-Seamless Tested 1000 lb. Grade A." Each specimen was inserted 14 in. into the 28-in. long, $2\frac{1}{2}$-in. standard weight seamless tube sleeve, and was supported at points 48 in. from the ends of the sleeve on steel plates and rocker supports.

The specimens for the second test were $2\frac{1}{2}$-in., nominal diameter, standard weight, seamless steel tubing marked and supported as for the first test. The 28-in. sleeve was of 3-in. standard steam pipe.

The results of the two tests, and comparisons with the values recommended by the authors are presented in Table 6. An examination of the results of the

TABLE 6.—COMPARISON OF TEST RESULTS

Item No.	Description	Test No. 1	Test No. 2
1	Nominal diameter of specimen, in inches	2.0	2.5
2	Maximum load recorded on testing machine, applied at the center of the test set-up, in pounds	1 500	2 600
3	Maximum bending moment,* in inch-pounds	36 000	62 400
4	Section modulus,† S, in inches^3	0.563	1.062
5	Modulus of rupture, $f = M/S$, in pounds per square inch	64 000	58 700
6	Recommended modulus of rupture for use in design (see Table 5), in pounds per square inch	70 000	70 000
7	Recommended bending moment for use in design (see Table 5), in inch-pounds	39 200	74 900
8	Comparison of results, in percentages	8	16

* $M = \frac{1}{2}$ (Item 2) times 48 inches.
† The section moduli were those computed from the standard dimensions for the weights of pipe tested.

two tests shows that a departure from generally recommended design values is to be expected in individual cases. It is important that tests be made of the selected pipe when close control of the water level at failure of the boards is desired, with the thought in mind that a change in design may possibly be made, based on such test results.

The method of testing, using two specimens at one time, tends to give an average value for the modulus, although in the tests described herein the measured deflections showed that the 28-in. pipe sleeve moved downward always parallel to the bed of the machine, indicating the uniform strength of the two specimens.

Acknowledgment.—Irving Cowdrey, Associate Professor of Testing Materials, Massachusetts Institute of Technology, devised the method of testing and conducted the tests in the laboratories of the Institute; acknowledgment is made for permission to include a description of his rig in this discussion.

E. T. SCHULEEN,[10] ASSOC. M. AM. SOC. C. E. (by letter).[10a]—Considerable credit is due the National Hydraulic Laboratory for its tests on flash-boards and to the authors for their suggested methods of rational flash-board design. Apparently, the tests have been conducted in a thorough manner and the use of a full-size flash-board should increase the reliability of the data. It is to be regretted, however, that limitations of discharge in the laboratory precluded tests at higher overflows, particularly at the larger pipe sizes.

[10] Hydr. Test Engr., Pennsylvania Water & Power Co., Holtwood, Pa.

[10a] Received by the Secretary September 15, 1939.

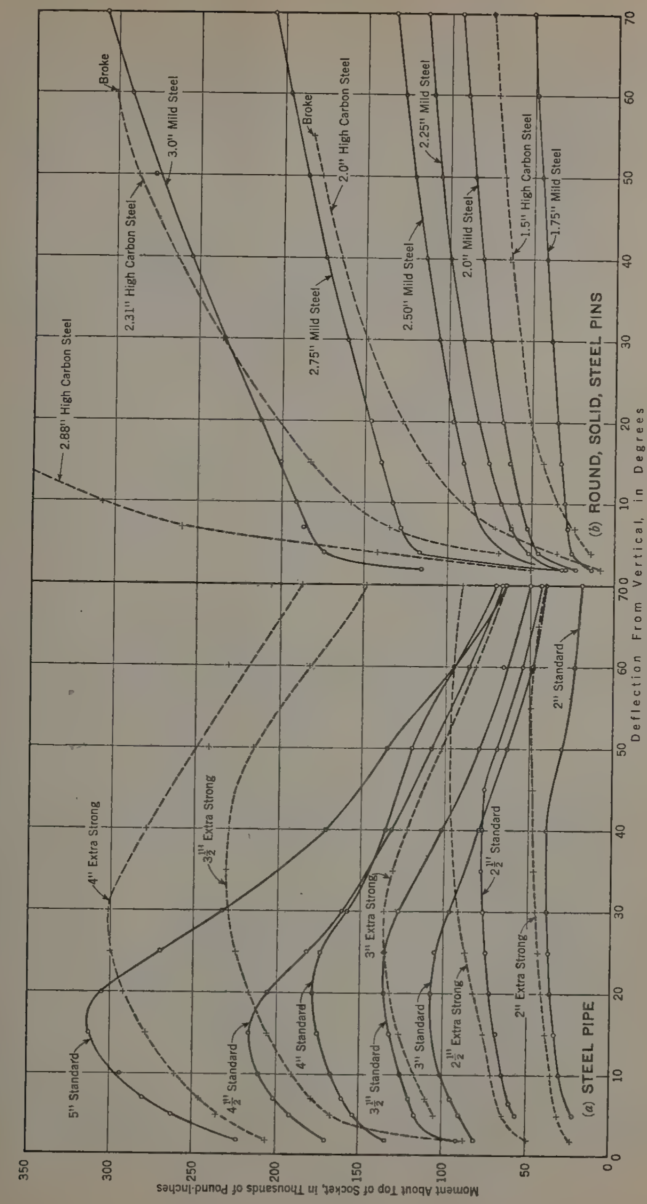

Fig. 14.—Results of Mechanical Bending Tests

The Pennsylvania Water and Power Company conducted a large number of tests several years ago on flash-boards and flash-board pins at its hydroelectric development at Holtwood, Pa. These tests included mechanical bending tests on solid steel pins, wrought iron pipe, standard steel pipe, and extra strong steel pipe of various sizes. Tests were also made to determine the actual water pressure exerted on a flash-board deflected through various angles from the vertical under several depths of overflow.

The mechanical bending tests were conducted in the following manner: The specimen under test was inserted in a socket of 6-in., extra strong, pipe 12 in. long. The socket was reinforced at the top with a pipe flange and grouted into a concrete floor. Suitable sleeves were used in the socket to give a snug fit with the various sizes of pipes and pins. The load or pull was exerted at a point 40 in. above the socket and was maintained normal to the specimen throughout the test by connecting the specimen, at the point of pull, through a spring-balance dynamometer to a 4-in. channel, which, in turn, was hinged to a support at the base of the specimen. A pull from a windlass was then exerted on the channel, causing it to rotate with the specimen. The pull normal to the specimen was read on the dynamometer, while the angle of deflection was read on a graduated scale. Since only the resisting moment at the top of the socket is required, the varying load distribution of the hydraulic pressure may be replaced by a single mechanical load such as was done in the test without introducing an appreciable error. Three specimens of each size of pipe or pin were tested and the results averaged.

The results of the tests on steel pipe are shown in Fig. 14(*a*) in which the resisting moment has been plotted against the angle of deflection. The writer feels that nothing is gained in reducing the resisting moment to unit stress by dividing through by the section modulus, which means dividing by a constant for a given pipe size. In addition, such procedure gives misleading stress values, as the actual section modulus is changing continually as deflection progresses. The curves in Fig. 14(*a*) show that pipe is well adapted for use as flash-board supports since the deflection increases with the moment until a definite angle, characteristic for each pipe size, is reached, beyond which the deflection increases while the moment decreases. This is a desirable characteristic which permits the pipe support, once the flash-board deflects to the angle of maximum moment, to bend over, completely and quickly, leaving little obstruction to the passage of water, ice, or debris. Table 7 summarizes the maximum moments and the angles of failure for the various sizes of pipe tests. The resisting moments of the individual test specimens of the same size varied within 5% of the averages given on the curves and in the table.

The Holtwood tests are comparable with the National Hydraulic Laboratory tests for pipe sizes of 2, 2.5, and 3 in. The angle of deflection at failure is greater in the Holtwood tests than the National Hydraulic Laboratory tests and the value of resisting moment is slightly less.

The results of some typical tests on solid steel pins are shown in Fig. 14(*b*). It is evident from these curves that the continued gradual increase in deflection with increasing moment makes solid pins unsuitable for flash-board supports,

.s very high forebay levels would occur before complete failure of the pins took Jlace. Another disadvantage is the tendency for the pins to bend from 5 to ; in. above the socket and thus continue to obstruct the passage of ice or debris .fter failure. It was found by test that these two difficulties could be overcome y cutting grooves on the front and back sides of the pin at the support, but his method is not practicable. The use of solid pins will usually be far more xpensive than the use of pipe.

TABLE 7.—Results of Bending Tests on Pipes

Size of pipe, in inches (1)	Maximum resisting moment, in pound-inches (2)	Deflection from the vertical, at maximum moment, in degrees (3)	Size of pipe, in inches (1)	Maximum resisting moment, in pound-inches (2)	Deflection from the vertical, at maximum moment, in degrees (3)	Size of pipe, in inches (1)	Maximum resisting moment, in pound-inches (2)	Deflection from the vertical, at maximum moment, in degrees (3)
(a) Standard Steel			(b) Extra Strong Steel			(c) Standard Wrought Iron		
2.0	39 000	40	2.0	48 000	53	1.5	25 000	50
2.5	77 000	44	2.5	97 000	48	2.0	40 000	40
3.0	107 000	20	3.0	135 000	30	2.5	74 000	40
3.5	136 000	23	3.5	231 000	35	2.5†	98 000	50
4.0	178 000	20	4.0	300 000	30	..	....	..
4.5	216 000	15	2.5*	201 600	75	..	....	..
5.0	312 000	17	..	....	..	..	....	..

* Double extra strong. † Extra strong.

The computation of moment on a flash-board with overflow after deflection ıas started, with a normal moment formula, such as Equation (9), is obviously n error since static pressures are attributed to flowing water. An attempt to letermine the magnitude of this error was made in a series of tests in Holtwood n which the actual moment of the board was measured at several conditions of overflow.

A test board 2.25 ft wide and 1.8 ft high was placed in a flume of equal width and was rigidly fastened at the bottom to a steel shaft free to rotate in ball bearings. On the outside of the flume, a cast-iron quadrant of 1-ft radius was keyed to the shaft and connected to a spring-balance dynamometer by a thin steel tape running over its edge. The dynamometer, in turn, was connected by a cable to a hand-operated winch. The quadrant and the board were counterbalanced so as to be in balance in any position. A seal of negligible friction at the edges of the board was provided by coating the walls of the flume with paraffin and scraping for close clearances. Four piezometers were placed in a vertical line on the board to indicate the pressure distribution. The board could be held at any angle under various heads and the moment about the bottom of the board, in pound-feet, could be read directly on the dynamometer. Tests on the 1.8-ft board were limited to four heads ranging from 2.2 ft to 3.3 ft.

The results of the tests are shown in Fig. 15, in which the measured moments, together with the moments computed from the normal moment formula,

are plotted against the angle of deflection. At 2.2-ft head the test moment increases with the computed moment up to a deflection angle of 30° and then decreases, while the computed moment continues to increase. At the higher

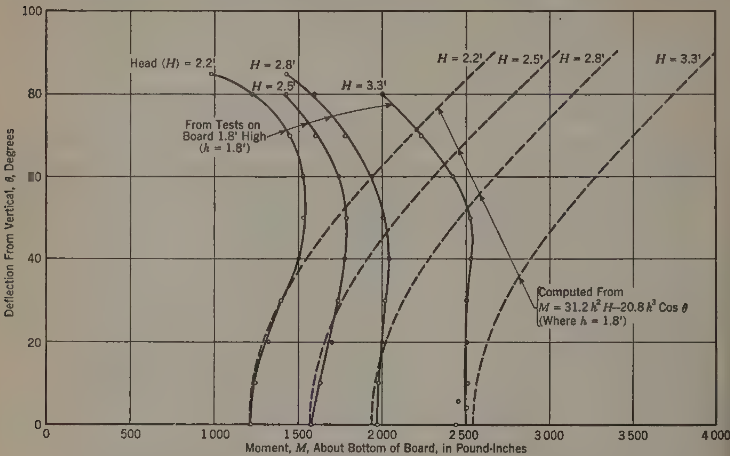

Fig. 15.—Comparison of Computed Moments with Actual Moments at Various Angles of Deflection

heads, the point of divergence between the test and computed moments occurs at lower angles of deflection, and at 3.3-ft head there is an appreciable difference at zero deflection. The piezometer readings indicated this reduction in pressure as the angle of deflection increased to be greatest near the top of the board.

An interesting corollary of the tests was the determination of the effect on moment of a vacuum on the down-stream side of the board. Since the test board was not aerated, a vacuum was found similar to that occurring on a long line of flash-boards. Breaking this vacuum by inserting a pipe through the nappe decreased the moment on the board approximately 7 per cent.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WIND BRACING IN STEEL BUILDINGS

SIXTH PROGRESS REPORT OF SUB-COMMITTEE NO. 31 COMMITTEE ON STEEL OF THE STRUCTURAL DIVISION

Discussion

3Y MESSRS. SAMUEL T. CARPENTER, AND ROLLAND A. PHILLEO

SAMUEL T. CARPENTER,[41] ASSOC. M. AM. SOC. C. E. (by letter).[41a]—The 'ommittee has done a remarkable piece of work in clarifying the differences f opinion that have heretofore existed in regard to the suitability of the antilever and portal methods. However, the Committee has not taken a clear tand on the question of design as divorced from investigation or analysis. n presenting the curves for the value of wind reactions, Fig. 2, and the Witmer nethod of K-percentages, the profession has been given a tool for analysis which is applicable only after a design is made. The practicing engineer needs design first.

The influence of girder stiffnesses on wind reactions has been realized and apitalized on by Mr. Spurr in his practical method of design[6] which, at the ame time, takes into account the flexibility and rigidity of the structure. Mr. purr contended that it was possible to design a structure elastically so that the ssumed wind stresses throughout would be achieved in the final structure. his was proved to be the case in the model studies made by the writer with rofessors Large and Morris. This work[16] showed that Mr. Spurr's method ould be trusted and it is the opinion of the writer that it is the only practical nd applicable method now available for design. The writer believes, therefore, at the Committee should have placed more emphasis upon the applicability the Spurr method and also upon the importance of controlling the rigidity of e structure. Furthermore, as was also stated by the Committee, no direct

NOTE.—The Sixth Progress Report of Sub-Committee No. 31, Committee on Steel of the Structural vision, was presented at the meeting of the Structural Division, New York, N. Y., January 19, 1939, d was published in June, 1939, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as lows: September, 1939, by Messrs. Arthur G. Hayden, Robins Fleming, and C. M. Goodrich.

[41] Asst. Prof., Civ. Eng., Swarthmore Coll., Swarthmore, Pa.

[41a] Received by the Secretary September 26, 1939.

[6] "Wind Bracing," by Henry V. Spurr, McGraw-Hill Book Co., Inc., New York, N. Y.

[16] *Bulletin No. 93*, Ohio State Eng. Experiment Station, Columbus, Ohio, p. 23.

comparison may be made between the portal and cantilever methods without a comparison of rigidities.

The Witmer method of K-percentages, and the apparent adaptability of a three-story compromise bent, may be an important addition to methods of analysis. The presentation of this method should have been accompanied by either a derivation or the complete underlying logic of the five fundamental steps indicated. In view of the increasing importance of evaluating secondary stresses due to column deformation, and the fact that the designer is dealing with a super indeterminate structure, not readily reduced with confidence to a compromise three-story bent, the writer believes the use of this method should be restricted to preliminary analyses.

Professor Large has contributed much in his complete treatment of the effect of column deformation on wind stresses.[11] It points to a clear reason for a greater use of the cantilever method in the case of slender bents. The vertical reactions at the base of the 55-story bent follow a cantilever distribution, although the girder shears of the individual floors in the lower stories differ from cantilever values. Since the bent was designed by the portal method it may then be said that only in these lower floors were the portal values approached. Professor Large has already indicated these facts. The writer would like to suggest that cantilever values could also be achieved here by elastic designing.

The proposed method of evaluating the torsional effects of wind on buildings is very interesting and important. The writer has investigated this problem hastily by a more complex method, involving a three-dimensional moment distribution, and the results were in all ways comparable. The writer suspected that the individual columns might be subjected to a substantial torsion; however, the torsion in each column was found to be negligible.

ROLLAND A. PHILLEO,[42] ASSOC. M. AM. SOC. C. E. (by letter).[42a]—With reference to Part D, concerning the magnitude of the assumed wind force, it is gratifying to note that the Sub-Committee, in formulating the proposed wind load for the "standard" rather than the extreme condition, has recognized the impracticability of setting up a uniform provision for the entire country. It is suggested, however, that the Sub-Committee define (at least approximately) what it considers the "standard" condition to be in terms of maximum wind velocity. With the extreme range of highest recorded velocities that obtains within the continental United States, there is danger that the "standard wind load" (somewhat like the "average American") will be highly fictitious.

In previously calling attention to computed wind pressures ranging up to 110 lb per sq ft,[22] as noted in the report (see text following Fig. 12), it was not the writer's intention to advocate such excessive values for design loadings. In view of such data, however, it is believed that the increased loading recommended in this report is quite justified.

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[11] "Settlement Stresses in Continuous Structures," by George E. Large, *Bulletin No. 103*, Ohio State Univ. Eng. Experiment Station, Columbus, Ohio.

[42] Designing Engr., Pacific Railway Equipment Co., Los Angeles, Calif.

[42a] Received by the Secretary October 13, 1939.

[22] *Civil Engineering*, June, 1931, p. 870.

The writer has long been of the opinion that a design load of 30 lb per sq ft, which is specified in many present-day building codes, does not represent, adequately, the pressure which a wind of from 100- to 120-miles-per-hr velocity will exert on a building, particularly at elevations greater than 200 ft. For such conditions it is believed that Curve (3) of Fig. 12 (proposed by Mr. Wing) indicates the true pressure more accurately. This is not to say, however, that buildings designed for a load of 30 lb per sq ft are unsafe, for here enters the great bugaboo of the entire problem—that is, the strengthening effect of elements other than the frame. Such effects are generally difficult to evaluate, but it appears reasonable to assume that in some typical office buildings they may serve to reduce the effective pressure on the frame alone by as much as 50 per cent. It is probable that more than a few buildings that have been designed for wind loads of from 20 to 30 lb per sq ft have come through severe winds unscathed by virtue of this fact alone. The point to be emphasized in this connection is the danger that lies in writing a blanket specification in which the probable maximum wind pressure is reduced by a more or less arbitrary factor because some such modifying effects generally obtain.

Although it may not be strictly in line with the Sub-Committee's investigations, the writer would like to see some recommendation as to the permissible increase in unit working stresses for wind-stressed members, since this provision may have considerable effect on the economy of members carrying high wind loads.

The Sub-Committee is to be commended for its excellent work embodied in its Sixth Progress Report.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

SEWAGE DISPOSAL PROJECT OF BUFFALO, NEW YORK

Discussion

By C. A. Holmquist, Esq.

C. A. Holmquist,[2] Esq. (by letter).[2a]—The pollution of Niagara River by sewage and wastes from Buffalo and other municipalities has been a matter of grave and growing concern to the Sanitary Engineering Division of the State Department of Health ever since its organization in 1906. For many years, the polluted Niagara River has been the source of water supply for the cities of Lockport, Tonawanda, North Tonawanda, and Niagara Falls, and the villages of Lewiston, N. Y., Youngstown, and Fort Niagara, having a combined population of approximately 140,000. Years ago, and before water-filtration plants had come into common use, these communities suffered repeated outbreaks and an extremely high incidence of typhoid fever. Niagara Falls, for example, prior to construction of its water-filtration plant in 1912, had the highest typhoid-fever death rate of any city in the state, reaching a maximum of 185 per hundred thousand in 1905 and averaging 131.2 for the 12-yr period ending in 1912.

The installation of water-filtration and chlorination plants at Niagara Falls in 1912 produced the immediate effect of lowering the typhoid death rate greatly. There has not been a single typhoid outbreak in the city traceable to the public water supply since 1913, and there have been only two deaths from typhoid since 1925 from any cause. This naturally speaks well for the efficient operation of the city's water-filtration plants. The improvements made to the water supplies of other communities along the Niagara River produced similar effects on the typhoid-fever death rates.

A number of investigations of the pollution of the Niagara River by the Department of Health have shown that the sewage and wastes discharged into the river from Buffalo have a tendency to become stratified laterally and to "hug" the easterly shore of the easterly channel of the river. This condition naturally has afforded considerable protection to the water intakes which are

Note.—This paper by Samuel A. Greeley, M. Am. Soc. C. E., was published in October, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[2] Director, Div. of Sanitation, New York State Dept. of Health, Albany, N. Y.

[2a] Received by the Secretary October 25, 1939.

located near the westerly shore. This so-called "natural barrier" and the greatly reduced typhoid-fever death rates, as a result of water-supply improvements, have been advanced as arguments against the construction of sewage treatment facilities by the city of Buffalo.

Limitations in the Public Health Law made it difficult to act to require installation of sewage treatment facilities by Buffalo in the face of reductions of the typhoid-fever death rate which had occurred. Furthermore, industrial wastes were a large and important factor in the pollution of the Niagara River, and the Public Health Law exempts industrial wastes from application of the law unless they are mixed with sewage. Over the course of many years the State Department of Health had tried repeatedly, but without success, to have the laws amended to give it more authority to control pollution and to simplify the somewhat cumbersome procedures established for its enforcement. These were some of the difficulties which the Department of Health had always encountered in its repeated efforts to have Buffalo provide adequate treatment facilities.

The several investigations made by the Department of Health over a period of fifteen years showed clearly that pollution of the Niagara River was increasing, and that at times the lateral stratification of pollution in the river was not so effective a barrier as it was thought to be at one time. The Department of Health always recognized, notwithstanding the reduction in typhoid fever which had been achieved, that the situation was one which could easily lead to an explosive outbreak under the failure of water-treatment processes. It was for this reason that every effort was made to secure action by the City of Buffalo.

The entire matter of abatement of pollution along the Niagara frontier "came to a head" in 1931 when the governor was convinced by the state commissioner of health that the situation held great promise for a calamity in the near future. Accordingly, the governor called into conference representatives of all the Niagara frontier municipalities and requested them to show cause why orders should not be issued requiring the abatement of the dangerous pollution. Buffalo, represented by its mayor, commissioner of public works, and corporation counsel, admitted its responsibility for the pollution but claimed that it could not finance the cost of treatment works without greatly exceeding the limit of its borrowing power. The other municipalities along the river agreed in effect to install treatment facilities as soon as Buffalo proceeded with a solution of its problem (which was, of course, the major problem).

No definite action resulted immediately from this conference, although it served the good purpose of impressing upon Buffalo and these other communities that pollution of the river could not be tolerated much longer. The Depression occurred and it was impossible, for a time, to induce municipalities to make appropriations for public improvements.

Then in March, 1933, came a "wave of pollution" which swept down Niagara River and through Lake Ontario, affecting practically every water plant with intakes extending into either the Niagara River or Lake Ontario, and causing an outbreak of about 10,000 cases of gastro-enteritis in the City of Niagara Falls. This experience and the threats to the lives and health of

thousands of persons served to strengthen, greatly, the Department of Health's case against the City of Buffalo.

Funds were becoming available from the U. S. Public Works Administration (PWA) about that time, and persistent efforts were made to induce the city to take advantage of the opportunity of federal aid for construction of the necessary facilities. Still the city did not move toward solution of this problem except through the construction of some storm-water relief sewers.

Early in 1935 another wave of pollution swept down Niagara River, which again threatened the safety of many public water supplies. The state commissioner of health decided that positive action against the city must be taken, and accordingly he instituted proceedings against Buffalo early in 1935, which resulted in the prompt issuance of an order, approved both by the governor and the attorney general, and which contained the following conditions:

(1) That a consulting engineer be engaged by May 1, 1935;

(2) That the purchase of the land for the sewage treatment works be authorized by June 15, 1935;

(3) That detailed plans and specifications for the Bird Avenue storm drain be submitted to the State Department of Health for approval by September 1, 1935;

(4) That comprehensive plans for the interception and treatment of the entire city's sanitary sewage, and necessary storm-water relief sewers, be submitted to the State Department of Health for approval by October 1, 1935;

(5) That detailed plans and specifications for sewage treatment works and a sanitary sewer system be submitted to the State Department of Health for approval by April 1, 1936, and that work be started by July 1, 1936, and the treatment works completed and placed in operation by October 1, 1937; and

(6) That by October 1, 1935, the City of Buffalo submit to the State Department of Health, for approval, a satisfactory program for the construction of the remaining interceptors, pumping stations, force mains, and storm-water relief drains to provide for the interception and treatment of the sanitary sewage of the city, together with the percentage of storm-water flow to be treated by the sewage treatment works.

However, its existing indebtedness made it impossible for Buffalo to finance the works by ordinary methods. Accordingly, the city had legislation enacted providing for the creation of a sewer authority with power to take over the sewer system, issue bonds, and establish sewer rentals for the financing of the cost of the city's share of the project and pay for the operation of the works. The city received a 45% grant from the PWA, amounting to approximately $7,000,000.

Almost simultaneously the City of Niagara Falls undertook the construction of intercepting sewers and sewage treatment works, which were completed in 1939. It was necessary, however, to take steps to order the cities of Tonawanda and North Tonawanda to install similar works. With the installation of treatment works for these two remaining cities, the chapter of pollution of the Niagara River will be closed, and one of the most serious cases of pollution in New York State will have been abated.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

DESIGN OF A HIGH-HEAD SIPHON SPILLWAY

Discussion

By I. M. Nelidov, Assoc. M. Am. Soc. C. E.

I. M. Nelidov,[14] Assoc. M. Am. Soc. C. E. (by letter).[14a]—Not only does the paper demonstrate the design of a particular siphon, but it also contains a theory with general rules intended for the design. The author has based his statements on an ideal case of a frictionless siphon, consisting only of the throat section. For this case he determines the theoretical velocity based on full vacuum head, and the resulting discharge. He also classifies as "high-head" siphons those which have operating heads greater than 34 ft. Furthermore, he has selected a siphon with an 80-ft head of fairly uniform cross-sections, with the exception of the outlet section, which he throttled in order to fill the outlet leg; and he classifies it as a "high-head" siphon.

Under "Introduction" the author states: "The term 'high-head siphon' is used by the writer to denote siphons that operate under heads in excess of 34 ft, or the barometric height"; and, under "Upper Bend and Throat": "As a general rule experience with siphon spillways has established 24 ft of water as a maximum negative pressure to be allowed at any point in the structure."[3] It is not clear from the foregoing which of the two heads (24 ft or 34 ft) should be used in defining the high-head and low-head siphon.

Mr. Rock qualifies the conditions for an ideal frictionless siphon without outlet, as stated herein. This may be satisfactory for the purpose of design but is not acceptable for the purpose of checking when one must classify an existing siphon and obtain the discharge on the basis of this classification.

Referring strictly to terminology, one may ask why, for instance, a 50-ft head should be called a "high head." In reference to dams at which siphon spillways mostly occur it cannot be called a high head. Perhaps heads beginning with 100 ft or 150 ft should be called high heads.

Note.—This paper by Elmer Rock, Jun. Am. Soc. C. E., was published in April, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1939, by J. C. Stevens, M. Am. Soc. C. E.; and October, 1939, by Messrs. B. E. Torpen, G. E. Hyde, and R. B. Cochrane.

[14] Senior Engr. of Hydr. Structure Design, State Div. of Water Resources, Sacramento, Calif.

[14a] Received by the Secretary August 26, 1939.

[3] *Proceedings*, Inst. C. E., Vol. 231 (1930–31), p. 201.

The dividing line, in the opinion of the writer, lies not in the head but in the functioning of a siphon. To illustrate, a siphon with an operating head of 10 ft and in which there is a considerable contraction at the summit will create a limiting velocity; and a further increase of head more than 10 ft will not produce any increase of flow. Under the same head, if the maximum contraction is made at the outlet, the maximum velocity will not be developed. On the other hand, a siphon under 100 ft of head and with a maximum contraction at the outlet may not produce the limiting velocity; and, further increase or decrease of the operating head will produce a change in the discharge. If the outlet is sufficiently constricted this siphon will develop the maximum velocity.

Consequently, independently of value of the operating head, a siphon may flow with a limiting velocity and without an increase of discharge due to further increase of the head; or, it may flow with a velocity below the limiting value, discharge being affected by increase of the head. The writer proposes to divide the siphons into two classes depending upon whether the operating head is greater or less than the limiting head at which the siphon produces limiting velocity. Thus, siphons may be classed according to whether their discharge is equal to or lower than the maximum discharge. The head at which this division occurs may be called the "limiting head." It should be made clear that the limiting head should not be related to any definite value of 24 ft or 34 ft, but that it should be taken as the head producing the limiting velocity. As mentioned previously herein, the classification should be made general both from the standpoint of design and checking, so that by reviewing an existing siphon one may also be able to classify it.

As to the theory proper, it may be noted that if the postulates of the author about the position and magnitude of vacuum are temporarily accepted the following derivations will be produced.

The limiting head will be (see Equation (6)):

$$H_{\text{lim}} = (K_e + K_1\, n)\, h_v \qquad (14)$$

in which: K_1 and K_e are as denoted by the author; h_v is the velocity head at the throat corresponding to mean velocity, V_m; and n is a coefficient in the expression:

$$E_k = n\, h_v \qquad (15)$$

in which E_k is the average kinetic energy of the curvilinear flow per unit of flow, such that[15]

$$E_k = \frac{\int_0^d \frac{V^3}{2\,g}\, dy}{\int_0^d V\, dy} \qquad (16)$$

In order to understand, better, the conditions existing at the throat, and using the criterion that the absolute pressure at crest of the throat should not

[15] Discussion by Boris A. Bakhmeteff on "Tests of Broad-Crested Weirs," by James G. Woodburn, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 423.

be less than zero, one may draw Fig. 5. The notation is by Boris A. Bakhmeteff,[15] M. Am. Soc. C. E., and the author.

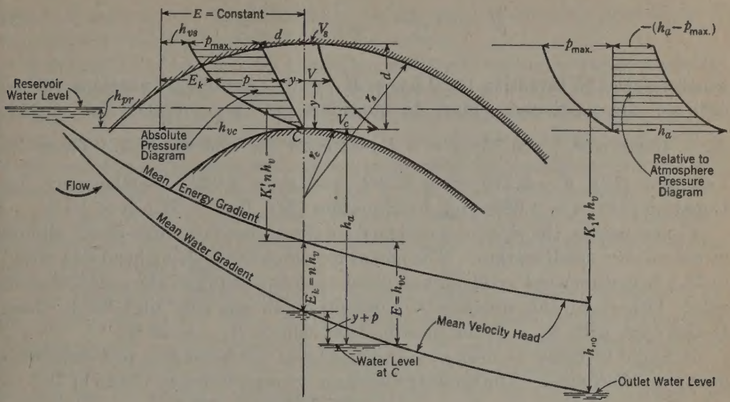

FIG. 5

Then, denoting $m = \frac{r_s}{r_c}$ and

$$\beta = \frac{V_s}{V_m} = \frac{1}{\frac{r_s}{d} \log_e \frac{r_s}{r_c}} \qquad (17)$$

integration of Equation (16) yields:

$$n = \frac{\beta^2}{2} \times \frac{m^2 - 1}{\log_e \frac{r_s}{r_c}} \qquad (18)$$

The criterion for zero absolute pressure at the crest of the throat is:

$$h_a + h_{pr} = h_{vc} + n\, h_v\, K_1' \qquad (19)$$

in which: h_a is the atmospheric head; h_{pr} is the priming head; $h_{vc} = m^2 h_{vs} = m^2 \beta^2 h_v$; and K_1' is the sum of the coefficients of loss from inlet to crest, referred to mean velocity of the throat. Substituting

$$h_v = \frac{h_a + h_{pr}}{m^2 \beta^2 + n\, K_1'} \qquad (20)$$

from Equation (19) into Equation (14):

$$H_{\text{lim}} = (K_e + K_1\, n) \frac{h_a + h_{pr}}{\beta^2 m^2 + n\, K_1'} \qquad (21)$$

The discharge per linear foot of the crest, with velocity $V_m = \sqrt{2\,g\,h_v}$, will be:

$$Q = d\,V_m = d\sqrt{\frac{2\,g\,(h_a + h_{pr})}{\beta^2\,m^2 + n\,K_1'}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

which reduces to Equation (4) if $h_{pr} = K_1' = 0$. Applying Equations (14) to (22) for the particular siphon in consideration: $K_1 = 1.035$; $K_1' = 0.154$; $K_e = 4.980$; $m = \frac{7}{4} = 1.75$; $d = 3$ ft; $h_a = 24$ ft; and $h_{pr} = 0$. Then, by Equation (17), $\beta = 0.770$; $\beta^2 = 0.593$; and $m^2 = 3.060$. Furthermore, by Equation (18): $n = 1.090$; and, by Equation (21), $H_{\text{lim}} = 78.5$ ft < 80 ft.

Consequently, the siphon as designed by the author is a "high-head" siphon by only a very small margin. The means by which the limiting head was raised to 78.5 ft is somewhat artificial and consists in throttling of the outlet section only. Otherwise, this siphon would definitely fall into the "high-head" class. In this case, with $K_e = 1$, the limiting head will be $H_{\text{lim}} = 26$ ft.

It would be better to designate this siphon as one belonging to a class with maximum discharge. The limiting discharge corresponding to a head of 78.5 ft, by Equation (22), will be: $Q = 3\sqrt{2\,g\,\frac{24}{3.050 \times 0.593 + 1.090}} = 3 \times 29.9 = 84$ cu ft per sec per ft. This is the maximum discharge that this siphon will produce, disregarding the further increase of the operating head.

In regard to the limits of applicability of Equation (4), it may be noted that with a constant depth of the throat "d" the discharge Q_1 decreases with the decrease of r_c, and does so especially rapidly for the ratio $\frac{r_s}{r_c} < 2$.

With a sharp crest such as may occur in a siphon with a vertical barrel, $\frac{r_s}{r_c} = \infty$ and, theoretically, the discharge approaches zero. Actually, however, there will be some flow through the siphon. A cavity will form above the sharp crest, producing some minimum radius r_c, depending on the conditions of the inlet. Consequently, if a siphon under consideration has an exceedingly small radius, r_c, it should be verified that the latter is greater than its allowable minimum if an expression for flow similar to Equation (4) is to be applied safely. The value of r_c (min) may be determined with the aid of the momentum principle used for entrance and throat sections.

The ratio of radii selected by the author is: $k = \frac{r_c}{r_s} = \frac{4}{7} = 0.570$. This is not the ratio, however, that gives the maximum discharge. From Equation (4) it follows that the discharge reaches its maximum value with a certain value of ratio k, r_s being constant. Representing Q_1 as a function of k:

$$Q_1 = 39.5\,r_s\,F(k) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

in which $F(k) = k \log_e \frac{1}{k}$. Differentiating F by k and equating it to zero, the

value $k = 0.367$ produces the maximum value of $F_{max} = 0.368$; and Q_1 (max) = 103 cu ft per sec per ft. In this case the radius at the crest should be $r_c = 0.367 \times r_s = 2.57$ ft, and the depth of the throat should be $d = r_s - r_c = 4.43$ ft, against $d = 3$ ft, selected by the author. It would be interesting to compare the author's basis for selecting the radii.

The position of the maximum vacuum within the siphon, so far as it may be determined for siphons with straight outlet leg, is at the crest of the throat section.[3, 16] In the siphons with another convex-upward curvature, besides the throat proper, situated down stream from the latter and without throttling at the outlet end maximum vacuum is developed at the section somewhat down stream from the throat section.[17] This is due to an unrestricted increase in velocity and could be relieved by throttling the outlet section. Therefore, apparently the position of maximum vacuum at the crest is justified for a siphon with straight outlet leg.

The magnitude of the maximum relative negative pressure is limited, by the author, to 24 ft. This value acquires significance for computing the "limiting head" and the maximum discharge. It becomes especially important for siphons at high elevations. For instance, at an elevation of 6 000 ft the vacuum head is normally reduced from 34 ft to 27 ft. If, then, 10 ft should be deduced from it for vapor pressure and other imperfections, it will leave only 17 ft as actual vacuum head. It is of importance, therefore, to make the recommendation as to the limit of the vacuum head to be used at different elevations more specific.

Prof. A. N. Gibson[3] states: "With reference to the limit of velocity at the throat of the siphon the maximum velocity should not be higher than would cause a negative pressure greater than about 24 feet of water." Mr. A. H. Naylor, one of the discussers of Professor Gibson's paper, states: "The pressure was lowest on the inside radius and that minimum pressure must not be less than a vacuum of about 28 feet of water." B. A. Etcheverry,[18] M. Am. Soc. C. E., states: "In practice on account of the air entrained in the water, the maximum suction lift is often taken as about 28 feet."

It follows from the foregoing citations that recommendations for vacuum head vary from 24 ft to 28 ft. On the other hand, during experiments made by J. C. Stevens,[17, 19] M. Am. Soc. C. E., negative pressures of about 32.5 ft were actually observed at Elevation 700 above sea level, all of which shows the variety of numerical values suggested for vacuum head. There is a need for a more precise definition of the vacuum head based more on experimental data than has been produced to date. Consequently, it would be valuable to receive from the author a more complete description of data upon which he recommended 24 ft of vacuum head with such confidence.

[16] "Hydraulic Laboratory Practice," by the late J. R. Freeman, Hon. M. Am. Soc. C. E., pp. 515 and 610.

[17] "Siphons as Water-Level Regulators," by J. C. Stevens, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 104 (1939), p. 1787.

[18] "Irrigation Practice and Engineering," by B. A. Etcheverry, and S. T. Harding, Members, Am. Soc. C. E., 2d Ed., McGraw-Hill Book Co. Inc., New York, N. Y., 1933.

[19] "On the Behavior of Siphons," by J. C. Stevens, *Transactions*, Am. Soc. C. E., Vol. 99 (1934), p. 986.

In conclusion, the writer wishes to say that the value of the proposed theory should be proved in checking the computed discharges with the discharges actually observed. He has selected the papers[17, 19] by Mr. Stevens for this purpose, using the observed maximum negative pressure at the crest as the velocity head h_{vc} (omitting losses from the entrance to the throat).

Siphons Nos. 5, 6, and 7, reported by Mr. Stevens, gave the discrepancies from the observed discharges as 0, −2, and 12 per cent. Siphons Nos. 1 and 2 reported in a second paper indicated a discrepancy of −6% and 0 per cent. This indicates that, at least for this type of siphon, the agreement of theory and practice is fairly satisfactory.